WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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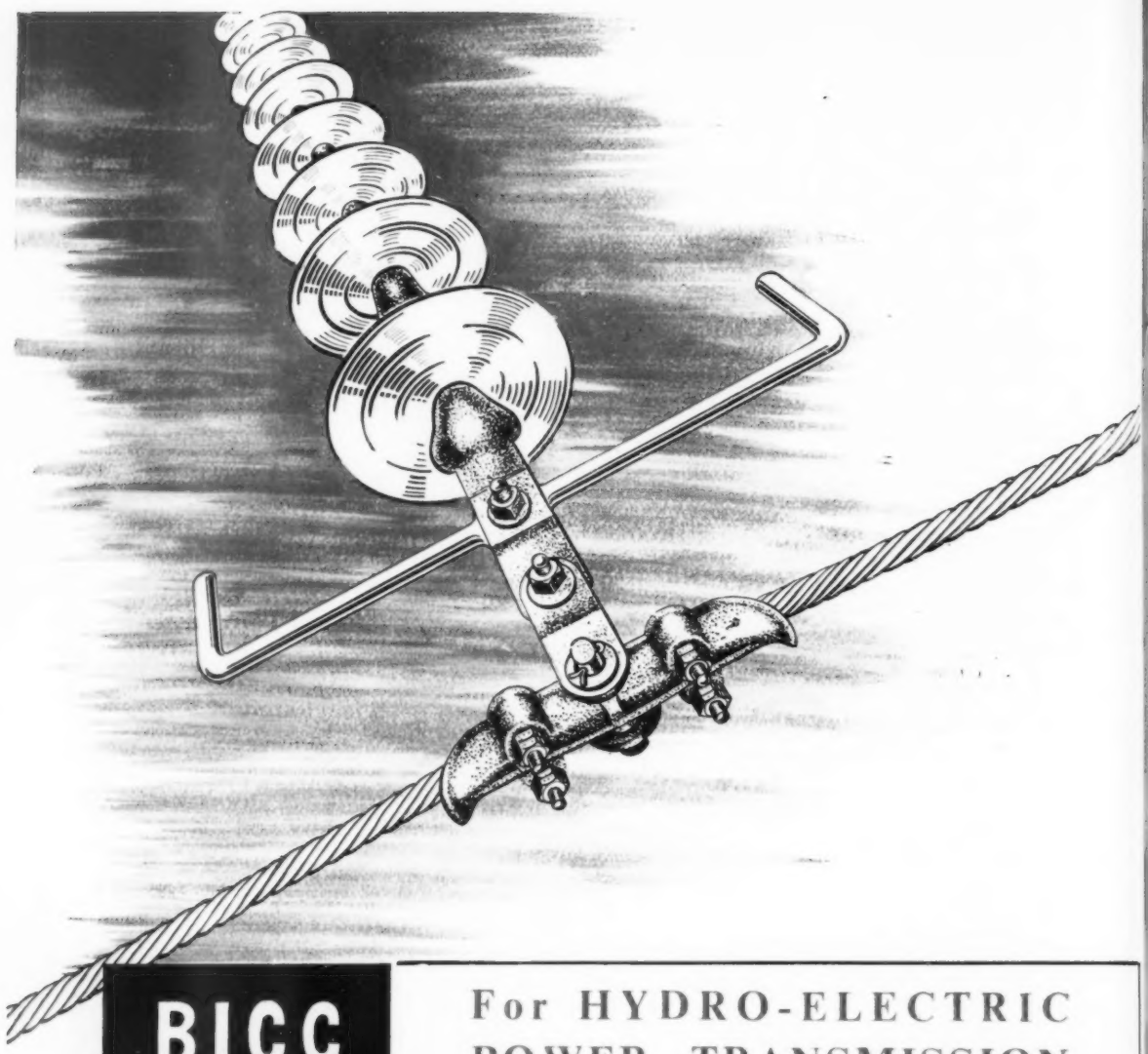

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WATER POWER

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The Nuclear-Hydro Relationship

WHEREVER power engineers meet, the discussion inevitably seems to turn to the possibilities of nuclear methods for providing the future needs of the world. We therefore make no apology for returning to this subject, which has already been mentioned in these columns on more than one occasion.

The two most advanced countries in regard to nuclear power generation are, of course, the United States and Great Britain. It appears that the first commercial-scale nuclear power plant to be commissioned will be that situated at Calder Hall, in Cumberland. It is likely to be generating power in the late summer or autumn of this year, and will be operated by the United Kingdom Atomic Energy Authority. A second nuclear power plant is under construction at Dounreay, in the north-east of Scotland, but it may be several years before this reactor, which is of the breeder type, will contribute any energy to the British Electricity network.

The Authority responsible for the whole electricity supply in England and Wales, together with the North of Scotland Hydro-Electric Board, is very naturally considering the position which nuclear power generation will occupy in its future plant programme, and many of the facts on which its decisions will be made will relate to the situation of nuclear power plant vis-à-vis hydro power and thermal power in other countries.

The basic problems to be faced in initiating a programme of nuclear power-station developments include the necessity to supply cooling water to almost exactly the same order of magnitude as that required for thermal stations; and when it is realised that 100 million gallons an hour is a reasonable figure for the sort of plant capacity nowadays envisaged, it will be seen that the choice of site is very considerably limited by natural conditions. Sea water does not form the best answer to the cooling-water problem, as tidal difficulties are often acute and the cost of the works necessary to bring in large volumes of water from well below the lowest tide level, and then to pump it up to the power station, which must inevitably be above the highest tide mark, tends to be prohibitive.

Next, we may consider the safety problem. It has recently been authoritatively stated in Great Britain that the danger of an explosion in a nuclear power station is extremely remote, and indeed according to some authorities, impossible; but it is admitted by the Atomic Energy Authority that the possibility of the escape of small quantities of highly radioactive material still exists. Such an escape would mean, we are told, that the whole of the population for about a quarter of a mile radius on the down-wind side of the power station would have to be evacuated for at any rate some hours. This consideration naturally limits the choice of site still further, since it effectively prohibits the establishment of nuclear power stations in the vicinity of large cities or industrial establishments.

Another problem which has to be solved when nuclear power plants take their part in power generation on a large scale, is the fact that, as far as can be ascertained from the most reliable sources, they will have to run at as near 100 per cent. load factor as possible. This will mean that the thermal plants with which they are associated may very well run at reduced efficiency, since a steam power station, involving a small number of very large generating units, must run at a high load factor to be efficient, and indeed the amount of coal that is wasted when such a station runs at anything below full load amounts in the aggregate to a very considerable figure each year. Thus the aim of the load despatching engineers is to give such plants every possible opportunity of running under base-load conditions, leaving the peaks to be taken up by less efficient plants. In this matter we may emphasise that in the vast majority of hydro-electric plants these considerations do not apply.

With regard to costs, both in regard to capital and running costs, we have been told by many engineers concerned with nuclear power generation that whereas with steam plant the capital cost is low and the running cost high, with nuclear power plant, the capital cost will be high and the running cost low. However,

the first estimates of cost for the Calder Hall plant, not yet officially confirmed but nevertheless freely mentioned, are that the capital cost will be of the order of £300 per kW, and that the running cost, first estimated as being of the order of 1d. per unit, will be reduced perhaps to 0·7d. per unit or even 0·6d. This compares with the overall figure for Great Britain for last year of 0·5594d. per unit (practically all steam plant), and it is hoped that improved plant efficiency will bring this latter figure down appreciably in the next few years. The capital costs in Great Britain for steam power stations are of the order of £60 to £65 per kW.

With all these considerations in mind, it becomes obvious that the linking together of hydro power and nuclear power will be much easier than the linking together of steam power and nuclear power. The loading problems, for example, almost entirely disappear, providing adequate transmission capacity, linking all the stations and the centres of load, is available. In exceptional circumstances, a given hydro plant may even have to run to a schedule to prevent unwanted water levels appearing in a river-fed reservoir, but in general, adequate daily regulation is always available, and the hydro plant could join the nuclear plant in the daytime when the load is heavy, leaving the nuclear plant to take all the load, to preserve its full load factor, during the night. The siting problem in regard to cooling water and the safety problem might well be solved by installing the nuclear power stations at the same sites as those where hydro power is developed; these are generally remote from centres of population, and suitable cooling-water facilities are obviously available. In addition, the transmission facilities could be shared, and certain of the operating staff might well function for both stations, thus showing a saving. The living quarters and other amenities provided for the hydro staff could, in many cases, be extended easily to take in the nuclear power-station engineers.

The capital and running-cost structures of the two types of power generation—hydro power and nuclear power—have much more in common than those of steam power and nuclear power. This may well be beneficial, since both methods of power generation lead ultimately to an enhanced usage of power which would be cheap and abundant, whereas power from thermal stations tends to be more and more expensive as coal gets scarcer, has to be transported over greater distances, and costs more to mine.

Hydro Power for Heavy Water?

A PARTY of British scientists is visiting New Zealand with a view to discussing the practicability of manufacturing heavy water with energy derived from water power. Three departments of the New Zealand Government will be involved in these discussions—the State Hydro-Electric Department, the Ministry of Works, and the Department of Scientific and Industrial Research. The talks will cover the provision of the necessary power, the detailed planning of the plant for heavy-water production and how best to deal with such incidental problems as corrosion. It is understood that Dr. H. R. C. Pratt, who visited New Zealand last year as officer-in-charge of the heavy-water project at Harwell, will be one of the party, while the other members will include two consulting engineers.

New Kariba-Kafue Report

THE French consultants who were appointed to examine these schemes have now reported that both are practicable and have given an estimated cost. This report is now being examined by the Government of S. Rhodesia who are also considering the availability of finance. Sir Godfrey Huggins, the Federal Prime Minister, has stated that a wrong decision on these schemes would have grave repercussions on the economic development of the country and that no decision could be made until there was a clear picture of the financial aspects. A mission from the International Bank is now awaited.

Ffestiniog and Rheidol Schemes

A BILL for the North Wales Hydro-Electric Power scheme has now been laid before the British Parliament and outlines the proposals which the British Electricity Authority put forward for the development of two hydro-electric stations. Main details of the scheme were given in a note which appeared in our November 1954 issue, page 406, but it may be recalled that the Ffestiniog station would be of the pumped-storage type and would involve the construction of two dams, separated by a vertical height of about 1,000 ft. This station would contribute some 300 million kWh of peak current to the 275 kV supergrid, while the Rheidol plant would supply 83 million kWh per annum, part of which would be distributed locally and the remainder absorbed by the grid.

New Tunnelling Record

IT is claimed that a new European record has been set up in the tunnel being driven in the St. Fillans section of the Breadalbane project of the North of Scotland Hydro-Electric Board. This tunnel is 10 ft. 6 in. diameter, driven through very hard epidiorite rock, and one face was cut to a length of 428 ft. in seven days. Taking all faces into account the tunnelling rate is now from 1,200 to 1,400 ft. per week; actual underground work was begun in August, 1954, and up to December 12 last some 8,500 ft. had been driven.

Mitchell Construction Limited were awarded this contract and are using an Eimco 21 Rockershovel to shift the spoil. This equipment operates on the overhead principle and is stated to have a sustained capacity of two tons per minute, the loading cycle being completed in seven seconds.

Sir Murdoch McDonald and Partners are the consulting engineers.

Glen Shira Scheme

THIS scheme, on which good progress is being made, will be the next to be put into operation by the North of Scotland Hydro-Electric Board. It comprises two power stations each of which uses the same water, the upper plant having a capacity of 5,000 kW and the lower 40,000 kW. The upper station at Sron Mor is of the pumped storage type and is of 5,000 kW capacity. This represents the first pumped-storage scheme that the Board has put into operation and indeed is the first of its kind in Great Britain apart

from a small industrial plant on the Tweed. The major station has a capacity of 40,000 kW in one unit and is operated from an inclined pressure shaft instead of the normal surface pipeline. The total fall utilised is 1,098 ft., the upper stage utilising 138 ft. and the lower stage 960 ft.

It should be understood that the main water storage is created by the upper-stage dam, but in wet weather it is expected that sufficient water from various collecting points will reach the smaller lower reservoir to feed the main station without drawing off from the high-level reserve. In periods of low load, say, at weekends, the output of the lower station will not be required for external use and will thus be available for operating the pumped storage system supplying the main high-level reservoir.

John Day Project

WE learn from the "Electrical World" of November 29, 1954, that the Pacific Power & Light Co., Portland General Electric Co., and Washington Water Power Co. have offered $350,000 to the Army Corps of Engineers to permit an immediate start on field exploration and preliminary design for the John Day Dam and power house on the Columbia River. The three firms explain that their offer has "no strings attached," and was prompted by the grave concern they feel over the pending power shortage in the Pacific Northwest which will develop early in the 1960's if no new large projects are brought into operation before 1963. The Corps of Engineers has not yet decided whether to accept the offer, which veteran Engineer officials in Washington call unprecedented, although they can do so under a 1925 law.

Ontario Hydro's Progress

A REPORT received from Mr. Robert H. Saunders, chairman of The Hydro-Electric Power Commission of Ontario, calls attention to the fact that since 1945 the capacity of the Commission's plant has increased by 84 per cent. During 1953 the dependable peak capacity of the Commission's resources was increased from 4,495,100 h.p. to 4,779,300 h.p., making a net yearly increase of 284,200 h.p. or 6·3 per cent. This additional power was mainly derived from new generating facilities provided at the Otto Holden, Richard L. Hearn and J. Clark Keith stations. During the year under review (1953) some 20,912·5 million kWh were produced for load purposes, this figure representing an increase of 4·7 per cent. on the previous year.

In the course of the report reference is made to the Sir Adam Beck No. 2 Station at Nigeria: the power station and associated works have proceeded on schedule but the design is now being modified to provide for a pumped-storage scheme and for a further four units to be erected to the main power house. This 1,828,000 h.p. project now calls for the installation of twelve units and the subsequent installation of six reversible pump-turbine sets to transfer the water to and from the storage reservoir. This part of the scheme will not be completed until 1957.

Torr Achilty Starts Generating

THE first of the two 7,500 kilowatt turbo-alternators in the North of Scotland Hydro-Electric Board's new power station at Torr Achilty in the Conon Valley, Ross-shire, were turned round for the first time on the 11th December and the second generating set is in course of erection and will be in production before next summer.

The Torr Achilty station is part of the Glascarnoch-Luichart-Torr Achilty scheme, the second stage in the hydro-electric development of the Conon Valley. The 2,400 kW Luichart power station, also part of this scheme, was brought into production in September, 1954, and another two power stations, Glascarnoch (24,000 kW) and Achanalt (2,000 kW) are now in course of construction.

This first turbine at Torr Achilty was turned round in the presence of Mr. T. Lawrie, C.B.E., and Mr. A. A. Fulton, respectively General Manager and Chief Hydraulic and Civil Engineer of the Board, Mr. J. B. B. Newton, Superintending Civil Engineer of Sir Alexander Gibb and Partners, civil engineering consultants, and representatives of Messrs. Merz & McLellan, electrical consultants, and Glenfield & Kennedy Limited, Kilmarnock, who built the turbines for Boving & Co. Ltd. of London.

Electricity Supply Conference

SEVERAL hundred representatives of the electricity supply industries in the principal European countries will visit hydro-electric schemes in the North of Scotland in the autumn of this year. The visit will follow the 10th Congress of International Union of Producers and Distributors of Electrical Energy (UNIPEDE), which is to be held in this country for the first time this year.

The working session of the Congress will last from 19th to 23rd September and be held in London. There will be two study tours in Scotland. The North of Scotland tour will include visits to hydro-electric schemes in the Tummel Valley, Glen Affric, Glen Garry and Glen Moriston. The delegates will be given a civic reception in Inverness. The second tour will include a visit to the Loch Sloy scheme, Portobello power station and the Borders. Before returning to London, all the delegates visiting Scotland will gather in Edinburgh, where a civic reception will also be given.

Rural Electrification Problems

THE United Nations Economic Commission for Europe reported recently on progress in the study, on an all-European basis, of rural electrification. Provisional reports were considered on a variety of subjects ranging from the operation of wind-driven power plants in parallel with low-capacity hydro and thermal plants, to the use of ultra-violet rays in agriculture to stimulate growth, counteract rickets in young animals and birds, and to promote egg-laying.

Preliminary discussions also took place concerning methods of financing rural electrification plants and the study of the various national safety regulations for electrical installations on farms, the latter aiming at the formulation of general recommendations to serve as a possible basis for future installations in the less highly industrialised countries of Europe.

The bulk of the information contained in the provisional studies was approved and the reports will be put into definitive form, with the addition of the new data submitted during the discussion.

Book Reviews

Handbook of Hydraulics (4th Edition). By Horace Williams King and revised by Ernest F. Brater. Published by McGraw-Hill Publishing Co. Ltd., McGraw-Hill House, 95, Farringdon Street, London, E.C.4. 7¾ in. × 5 in. × 1 in. 822 pp. 671 ff. Price 57s. 6d. net.

No book of this kind can receive a stronger endorsement of its worth than a demand for a further edition. Professor King did a notable piece of work in compiling this time-saving reference book, and in the present edition, although all the basic data and fundamental tables have been retained, the scope and value of the book have been profitably enlarged by Professor Brater's revision. A discussion of viscosity and numerical values of viscosity has been added, together with the results of recent experimental work on flow through orifices and culverts. The section on flow over submerged weirs has also been brought into line with the latest research findings. Other improvements concern the treatment accorded to non-uniform flow in open channels, and equations for laminar flow in open channels have also been included. Other amendments of a similar nature have been made throughout the book, and are all well calculated to increase its general excellence.

This handbook has always been a miracle of compactness, and not the least of its value to the busy engineer is the clear manner in which the methods of applying the data to the solution of practical problems are described.

A Subpressao nas Barragens (The Uplift in Dams). J. Laginha Serafim. Paper No. 55. Laboratorio Nacional de Engenharia Civil (Ministerio das Obvas Publicas), Lisbon. 243 pages, and numerous illustrations.

The Civil Engineering Research Laboratory in Lisbon, of which Mr. J. Laginha Serafim is chief engineer for research on dams, is interested in all aspects of civil engineering developments from housing and industrial developments to irrigation, coastal protection, harbour design and hydro-power schemes. The standard of the research work done in the hydraulic research section compares with anything done elsewhere, and some pioneering work has been done on the design and calculations of large arch dams (Santa Lucia, Castelo do Bode and Cabril dams). The Portuguese learned from the Italians how to build dam models and how to test them, and did some important research work of their own. The present publication is part of this work.

A study is carried out of the forces resulting from the entrance of water under pressure in the body of concrete dams and their foundations. The most important of these forces is that equivalent to a vertical buoyancy causing tensile stresses, which is generally called "uplift." The horizontal stresses in the dam are also affected, owing to the pressure exerted on the internal surfaces of the concrete.

In chapter 1 of the publication the general aspects of the uplift problem are pointed out, and analysed in chapter 2, starting with the theories of Maurice Lévy (1895), Fillunger and Hoffman. It is Terzaghi who showed that the effective area on which the uplift is acting is a physical characteristic (" boundary porosity ") of the concrete irrespective of the volumetric porosity. The value of this factor, very close to unity, can only be obtained through laboratory tests. Tests made by Leliavsky Bey and R. Davis supplied good value for this boundary porosity.

Chapter 3 establishes the true significance of boundary porosity, which can only be realised when there are variations of the pore pressure from point to point. The differential equations of elasticity (equations of equilibrium, boundary and compatibility conditions) are established.

The differential equation that regulates the unsteady flow of water, assuming that the law of Darcy is valid, is established in chapter 4. Reference is made to the Laplace equations relative to steady flow.

Chapter 5 discusses the research work from different sources in connection with flow of fluids in porous media with very narrow capillaries (compact concrete), and chapter 6 deals briefly with the problem of the initial drying of the concrete. Reference is made to the various trials for the measurement of the pore water pressure in dams, of which those undertaken by the T.V.A. on the Hivasee and Fontana dams were carried out with great success. In the same chapter it is shown that the movement of the water in rock foundations is generally very similar to that in homogeneous porous bodies.

Further tests carried out for the determination of the boundary porosity of mortar and hydrated cement paste are presented in chapter 7. Chapter 8 is a summary of the conclusions set out in the other chapters.

International Electrotechnical Vocabulary. Published by the Central Office of the International Organisation for Standardisation, 39 route de Malagnou, Geneva, Switzerland. Issued in three parts. Price: Fundamental Definitions (Group 05) 8 Swiss Fr., Machines and Transformers (Group 10) 8 S. Fr., Rules for Short Circuit Conditions (Publication No. 56) 10 S. Fr. Postage extra.

The International Electrotechnical Commission came into being in 1910 but the first Vocabulary was not published until 1938. It is now quite out of print, and the question arose as to whether it should be reprinted as it stood or whether it should be completely revised and reprinted. The rapid progress which has been made in certain fields of electro-technology—especially in electronics, telecommunications and electro-acoustics—decided the Committee to adopt the latter course.

This work was begun in 1949 under the direction of General Wiener and it is hoped it will be completed by 1956. The second edition will comprise 22 groups, or parts, in which each definition is given in French and English and the term itself in the German, Spanish, Italian, Polish and Swedish languages. The U.S.S.R. National Committee has been entrusted with the publication of the work into Russian.

The definitions have been drawn up with the object of striking a balance between absolute precision and simplicity, the engineer thus being provided with a working term, in his own language, about which there can be no misunderstanding.

Each section is provided with a complete index in French, English, German, Spanish, Italian, Polish and Swedish.

Fig. 1. Artist's impression of the Birsfelden power station

Birsfelden Power Station

A description is given of a 115 MVA Swiss development constructed on the international section of the Rhine between Augst-Wyhlen and Kembs stations. Navigational requirements had a special bearing on both design and construction, and the work is being carried out with German agreement and collaboration.

BETWEEN Basle and the Lake of Constanz, the Rhine is an international river, Germany and Switzerland sharing the hydro power in proportion to the length of the river banks on their national soil. Some hydro power stations are shared by the two countries, others belong entirely to one country only, each case being decided according to international agreement.

The power station at Birsfelden is located upstream of the town of Basle, and just upstream of the river Birs, which flows in on the left bank of the Rhine. Forty-one per cent. of the river banks between Birsfelden and the power house of Augst-Wyhlen (built in 1909-12) are on German territory; thus, according to international agreement 41 per cent. of the contracts for machinery and civil works had to be placed in Germany; also 41 per cent. of the workmen had to be German, an arrangement which suited the Swiss because of difficult labour conditions in their country at the time when construction began. In the particular case of Birsfelden, the whole output of the power station is reserved for the Swiss market, a previous arrangement having reserved the whole output of the international power station of Allbruck-Dogern (also on the Rhine) for Germany.

The first designs for a power station to be built at Birsfelden were studied in 1919, but the final project, a difficult one from many aspects, was really started in 1942. Difficult financial and technical negotiations with the German authorities of the state of Baden led to an agreement on how the navigational side of the project was to be solved and financed.

A company was founded on September 4, 1950 with an issued capital of 30,000,000 Swiss francs, the

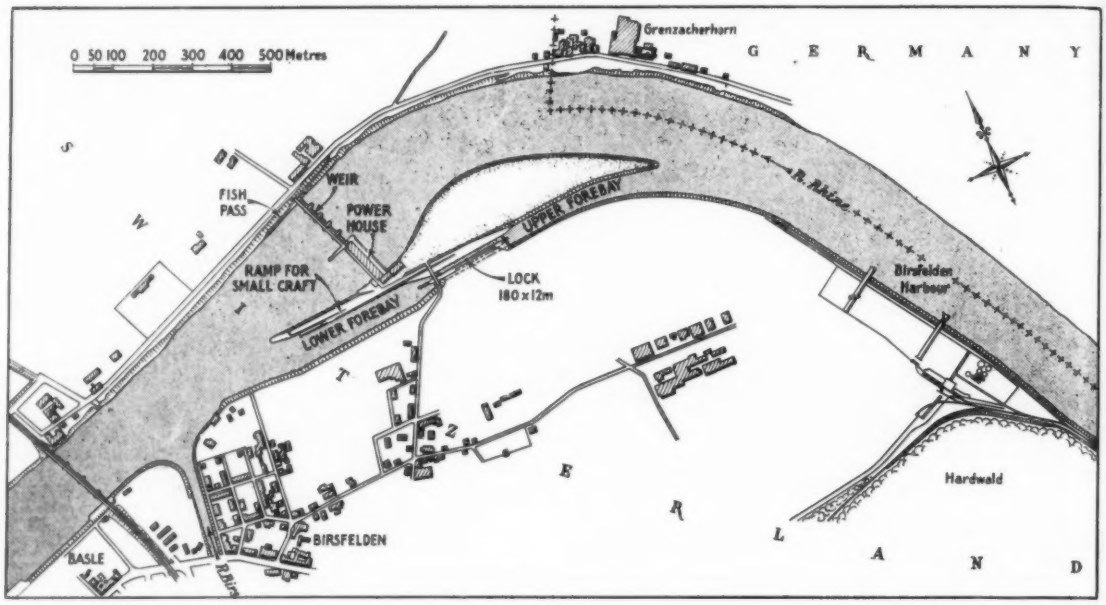


Fig. 2. General plan of the Birsfelden development

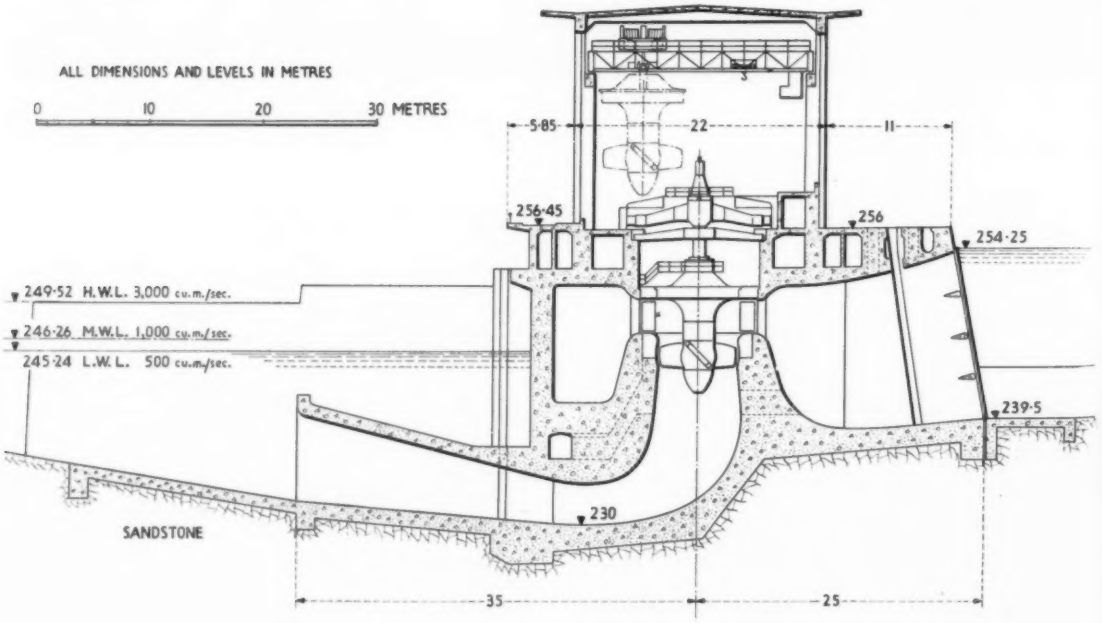


Fig. 3. Cross section through the power house

township of Basle holding 50 per cent. of the shares, the other 50 per cent. being subscribed by the Canton of Basle and two other private companies.

General Arrangement of the Station

The Birsfelden power project has been designed to utilise the hitherto unexploited head in the 8·45 km. stretch of the Rhine between Augst-Wyhlen and Kembs. The head varies between 8·1 and 4·1 metres for flows of 500 cu. m. per sec. and 3,000 cu. m. per sec. respectively. The upstream end of the reach to be developed begins at the lower concession limit of the Augst-Wyhlen power station, 600 m. downstream of the Augst weir, and ends 50 m. downstream of the

junction of the Birs, near the upper concession limit of the Kembs power station.

The 15 m. weir, with five openings each 27 m. wide, and the power house, with four turbines, are situated across the Rhine on a bend of the river, with the weir on the right and outer side of the bend, the power house on the left and inner side. The navigation lock, with upstream and downstream access channels, is on the left bank in the chord of the bend some distance from the power house, thus leaving an artificial island between lock and power house. This island will later on be developed as a park and playground at the outskirts of the town of Basle. This arrangement and the position of the power house some 650 m. upstream of the junction of the Rhine with the river Birs were dictated by the topographical and geological conditions, also by the need for maintaining navigation on the river during the construction period, and of bypassing the outlet of the Birs with its dangerous sandbanks.

In the past fifty years the town of Basle has developed an important harbour for river navigation, located mainly downstream of the town. The French boundary makes further extension of this harbour downstream impossible, and a new important harbour has more recently been developed upstream of the town and of the Birs. Thus it was essential that the project should not only allow navigation to develop in the future upstream of Basle, but also permit the free passage of barges en route to the existing harbour upstream of Birsfelden during construction. It was decided that at any time during construction a free passage of 27 m. width had to be made for navigation, the velocity of the water through this passage being not higher and the conditions for navigation not worse than those at the middle bay of the nearest bridge over the Rhine at Basle, so model tests to a scale of 1/100 were carried out at the Hydraulic Research Laboratory of the Federal Institute for Technology (ETH), Zurich. The general arrangement of the power station is shown in Fig. 2.

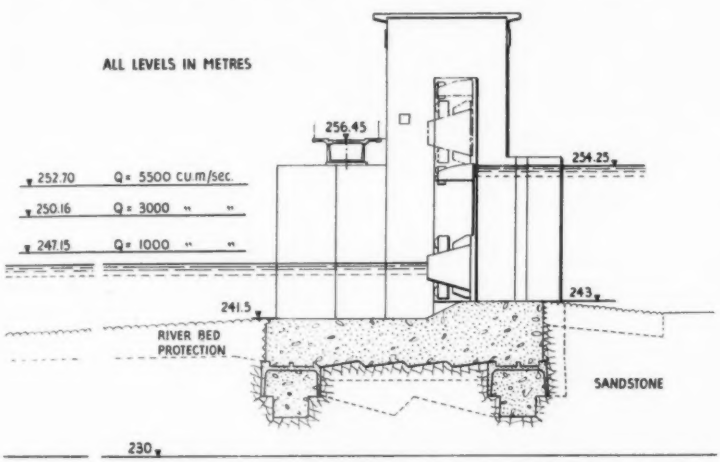


Fig. 4. Cross section through the weir and gates

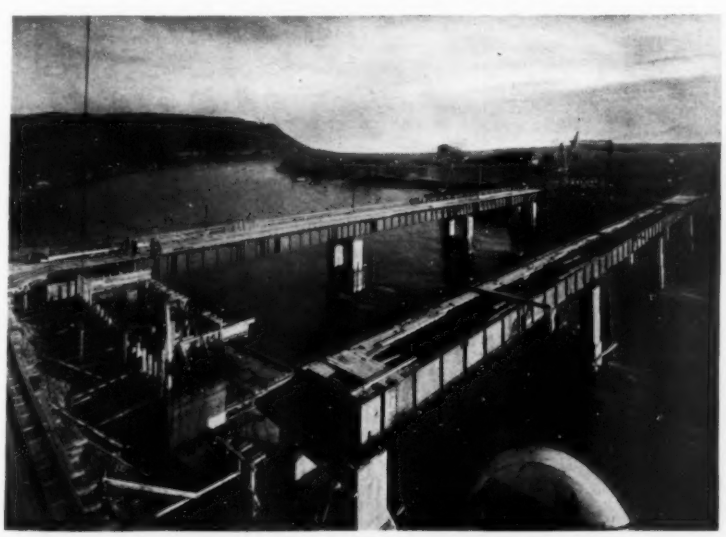


Fig. 5. View of site in December 1952 showing two temporary bridges

The Weir

The weir has been designed to permit the discharge of the greatest flow ever recorded (1876) of 5,500 cu. m. per sec., one opening being supposed closed for repair. This condition led to five openings of 27 m. width, and a total width of 157 m. For aesthetic reasons it was furthermore decided to dispense with any heavy, high-level bridge topping the weir, which would have spoiled the quiet river scenery near the junction of the Birs, a much-visited beauty spot near Basle. A footbridge only 2·20 m. above water level is being built on the downstream side of the piers to replace a Rhine ferry. This bridge is partly in steel, partly in concrete, and contains many prestressed elements. It is claimed that the weight of the bridge and costs have been reduced to a minimum. The foundations of the piers of the weir were carried to a depth of 10 m. below the weir sill. Model tests have shown that there is danger of erosion during flood periods, mainly downstream of the weir, and to a lesser extent upstream. So the river bed has been paved for protection for a length of 30 m. and to a depth of 2 to 3 m. thickness, with stone blocks of 500 kg. to 1,500 kg. This part of the project was also the subject of model study at the Federal Research Laboratory of Zurich. It was not possible to represent to scale on the model the sandstone which forms the bed of the river. From deep erosion caused by a bridge on the Rhine during a period of about ten or twenty years, it was presumed that the resistance

to erosion of the sandstone is low. The stone paving will reduce this erosion and remove it some distance from the weir.

There is no stone or coarse debris transportation in the Rhine at this point, as they have already been removed by the many reservoirs upstream. To determine the actual effect of the material carried by the river, abrasion tests were carried out with granite and concrete by rotating slabs of these materials in a machine with abrasive material for forty-eight hours (said to be equivalent to one hundred years of wear under normal conditions). The granite was practically unaffected, and only $\frac{1}{4}$ to $\frac{7}{8}$ in. of concrete was worn away. Thus it was considered unnecessary to protect the weir sill and piers with granite masonry, ordinary concrete facing being considered sufficient. This is the first time that a concrete facing alone for a weir sill has been used in Switzerland.

It was considered desirable for the gates to be sluice gates because of the navigation on the river. Each gate has two leaves, the upper leaf being a new type known as a "hook" gate, which consists of a steelplate skin, stiffened by a cross beam along its upper edge, and overlapping its bottom leaf, over which it can slide when being lowered. The lower leaf is of heavy base sections. Sector gates were also considered, but abandoned because the trunnions (as used at Rupperswill) required wider piers, as they protrude from the pier faces. The hook gate is only 4 m. high, and can be lowered in a few seconds by means of an oil brake. Each hook gate is connected to the governor of a turbine, so that when the turbine is closed suddenly, the hook gate is lowered automatically by a corresponding amount in order to maintain the discharge in the Rhine constant, thus avoiding dangerous positive translatory waves in the headrace and negative waves in the Rhine, downstream of the power station. Several gates may be lowered simultaneously as each is connected to one turbine only. The dimensions of the gate grooves are 4 ft. by $10\frac{1}{2}$ ft., and the rails 13 in. in breadth.

The hoisting mechanisms on each side of the gates for both leaves are located inside the piers. To synchronise the lifting movements on both sides of the gate the hoisting mechanisms are mechanically connected through a horizontal steel shaft passing below the footbridge. A diesel motor supplies the power for lifting or lowering the gates in case of a failure of electric power.

It was decided to dispense with the usual bridge for servicing the stoplogs on the upstream side of the gates, and five stoplogs will be lowered or lifted from two floating cranes, each of 30 tons lifting capacity. The stoplog grooves are approximately 4 ft. square.

The same floating crane can be used for lifting or lowering the stoplogs from the upstream end of the lock, or for any other duties. They are equipped with derricks and can easily be manoeuvred by means of their Voith-Schneider propulsion system. On the downstream side of the weir the weir openings may be closed by emergency arrangements. Special cavities

Fig. 6. Power-house foundations in August 1952

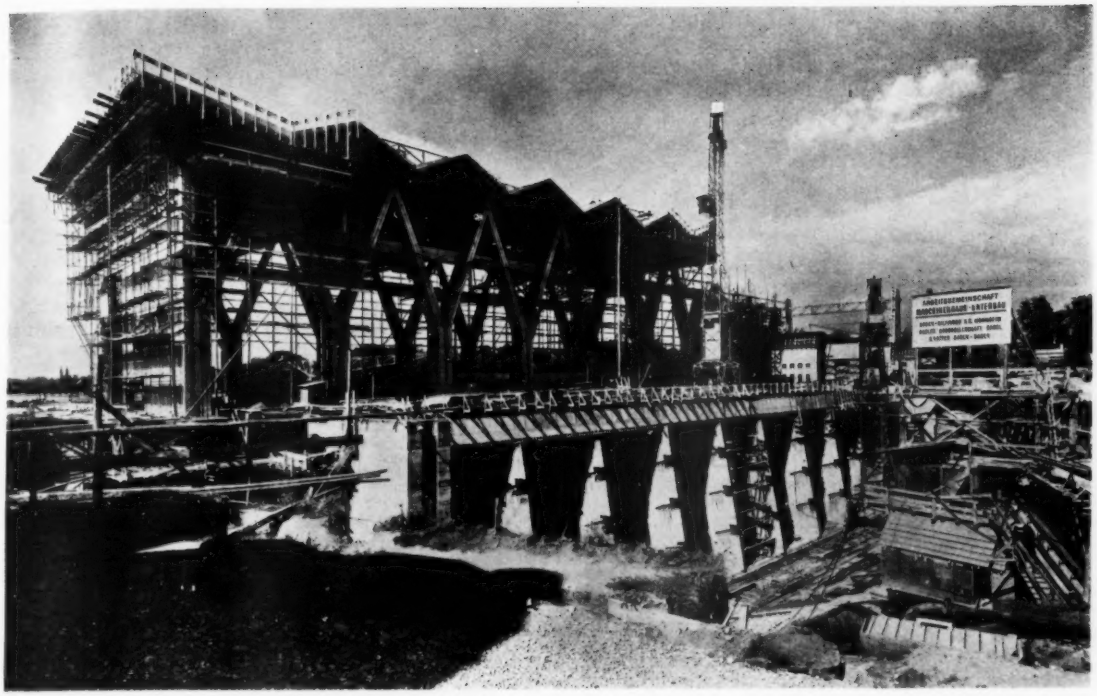


Fig. 7. Left half of power house during construction, showing turbine intakes and power-house superstructure

which are now filled with relatively weaker concrete may be broken open to take cross beams having cross bracing against the sides of the piers; then needles will be placed in front of these beams.

The Power House

The power house, 121 by 21 m. in size, is built on the left river bank, as a continuation of the barrage and at a slight inclination to it. It is separated therefrom by a bulky pier, which had an important function during the first stages of the construction programme.

The roof elevation of 280 m. is approximately 26 m. above the normal upstream water level, and the lowest point of the draught tube is at elevation 230·00 m., about 24 m. below the normal upstream water level. Most of the power-house foundations were excavated in the left bank on a width of about 100 m. The power-house inlet and the turbine outlet were tested on hydraulic models.

The architecture to be adopted for this power house was a major problem, as the power station is located just upstream of a major town, which has been for centuries a centre of arts and high culture, and it was felt that a conventional power-house design would destroy the quiet beauty of the scenery. The architects decided in favour of a very light building with wide glass panelling. The structure of the power house can best be seen in Figs. 1 and 3. Very remarkable are the Y-shaped pillars and the very light W-shaped power-house roof, for which prestressing has been used. When visiting the site last summer the power house was still under construction, as shown in Fig. 7. Nevertheless, it was possible, in spite of the scaffolding, to get a convincing impression of the architecture of the future building.

The power station is equipped with four Kaplan turbines of 30,000 h.p. 68·2 r.p.m., each coupled to a 28,600 kVA, 6,600 V generator. The turbines can absorb 325 cu. m. per sec. each. The speedring diameter is 7·2 m. and the internal diameter of the generator is 11·5 m. The downward thrust on the bearing is 900 tons. There are four draught tubes, each with a splitter wall, the leading edge of which is steel clad. Two of the draught tubes are of the rectangular Escher Wyss type, and the other two are of the oval Charmilles type. These were staggered, not from the hydraulic standpoint, but for ease of construction, one of each type being constructed together so that the shuttering could be used twice. A half-inch cork transverse expansion joint runs between Nos. 2 and 3 machines. An access gallery passes through the length of the power house with two access doors into each draught tube. The two cranes in the power house can lift 300 tons when coupled together. Upstream of the power house there are two travelling screen-cleaning machines. There are stoplogs for sealing both turbine inlets and outlets, which can be handled by the floating cranes.

The turbines and the power house (and also the navigation lock) have been designed to take full advantage of a future dredging of the river bed. At present navigation through and upstream of Basle follows a single navigation channel, which will later be enlarged to a double navigation channel. This will reduce the water level on the downstream side of the power house and increase the net head for the turbines.

The four transformers are near the turbines, in isolated cells, but can be handled by the main cranes. The 50 kV switchgear is located in a special switch house, 28 m. long, 14 m. wide and 13 m. high, and

housing three busbars, on the upstream side of and at right angles to the power house.

The control building is between the power house and the switch house and comprises the control room, workshop, staff room, sick bay and station-engineer's office.

Table I gives details of the possible energy production at Birsfelden. This table is based on an observation period of 20 years extending from 1923 to 1942. During this period the lowest discharge of the Rhine was 354 cu. m. per sec. and the highest flood reached 3,315 cu. m. per sec., the average discharge being 1,070 cu. m. per sec.

TABLE I. DATA CONCERNING THE POWER AVAILABLE AT BIRSFELDEN

	Low Water	Average Discharge	Design Discharge	Flood Discharge
Discharge cu.m./sec.	500	1,000	1,200	3,000
This discharge has been reached for.. days	340	178	123	0·1
Net head at Birsfelden m.	8·08	7·00	6·64	4·08
Power available at Birsfelden kW	33,600	57,600	62,400	27,120
Head loss at Augst-Wyhlen due to backwater m.	2·03	1·19	0·98	0·44
Loss of power at Augst-Wyhlen due to backwater kW	8,500	10,400	9,600	1,760
Net power at Birsfelden kW	25,100	47,200	52,800	25,360

The upstream water level, 254·25 m., has been chosen after careful consideration of the energy production possible and of the navigation requirements. When the water level at Birsfelden is at level 254·25 m., there is a loss of head at Augst-Wyhlen, the power station just upstream. This loss amounts to 2·03 m. at low water, to 1·19 m. at average water level ($Q = 1{,}000$ cu. m. per sec.), and 0·44 m. at high water level. A net loss of 78 million kWh of energy is thus entailed at Augst-Wyhlen, but there is a net gain of energy at Birsfelden. If the upstream water level at Birsfelden were reduced by say 1 m., the loss at Augst-Wyhlen would be reduced by 35 million kWh but the energy production at Birsfelden would be reduced by 63 million kWh.

The energy production in an average year at Birsfelden is given by Table II.

TABLE II. ENERGY PRODUCTION AT BIRSFELDEN IN AN AVERAGE YEAR

Yearly production (average)	440,000,000 kWh
Energy loss at Augst-Wyhlen	78,000,000 kWh
Net energy available at Birsfelden	362,000,000 kWh
In summer	200,000,000 kWh or 55%
In winter	162,000,000 kWh or 45%

Navigation

For many years the navigation of large vessels on this part of the Rhine has been considerable. The old navigation harbour extending downstream of the town of Basle cannot expand further north as it has already reached the political boundary with France, so a new harbour has been progressively developing south of the town, and in the year 1952 the turnover was near the 1 million tons mark. A smaller harbour is to be found on the right-hand German bank of the river. It is expected that the river traffic will increase in the near future. After long investigations, in conjunction with the Badish (German) government it was decided to design a lock capable of lifting large vessels or groups of large barges. The lock proper is 180 m. long and 12 m. wide; the lower forebay is 450 m. long (against only 250 m. for the locks farther upstream on the river), which is sufficient to moor four barges, and the width of 60 to 80 m. is sufficient for a large tug to turn and reverse. The usual width for a forebay on the Rhine is only 30 m. The upper forebay is 420 m. long and 54 m. wide.*

The time to fill or empty the lock may vary between 7 and 10 minutes, depending on the water levels. The total operation, including navigation through the forebays, takes 32 to 35 minutes going up and 21 to 25 minutes going down the river.

In addition to the lock a fish ladder, an incline for boats and an installation for canoes have been foreseen. Fish ladders 145 m. long are situated on the right bank (the sunny side of the river).

Cost Estimates

The costs of the hydro-power part of the project were estimated at 112,600,000 Swiss francs. The navigation part amounted to 23,775,000 fr., of which 16,155,000 fr. were charged against the power part and 7,620,000 fr. were to be paid by the German and Swiss governments and charged to the navigation part. (£1 = 12·4 Swiss francs.)

The detailed cost estimate (1948 price basis) shows the following figures:—

HYDRO POWER	Millions of Swiss francs
1. Weir	21·38
2. Power house:	
Civil-engineering contract	22·06
Electro-mechanical equipment	32·67
3. Switchgear building	2·91
4. River-bank protection, dredging of river bed	4·19
5. Fish ladder and incline for boats	0·80
6. Access roads, buildings, etc.	2·22
7. Project studies, financing costs, supervision, buying of land, etc.	26·37
Total	112·60
NAVIGATION	
1. Lock and lock bridge	7·00
2. Upper and lower forebays and dredging of the river	10·60
3. Projects studies, financing, supervision, etc.	6·18
Total	23·78
HYDRO POWER AND NAVIGATION	
Total	136·38

ESTIMATED ANNUAL EXPENSES	
Expenses for power	8,700,000 fr.
Expenses for navigation	1,150,000 fr.
Total	9,850,000 fr.
Government subsidies towards navigation	145,000 fr.
Final expenses	9,705,000 fr.

The production cost per kWh is estimated at 0·0268 francs. The output will depend widely on the river discharge.

Construction

The construction started on November 9, 1950, and it was expected that it would be possible to run the first turbine at the end of 1954. The construction pro-

* For comparison the lock at Donzere-Mondragon is 195 x 12 m. and "the international barge" is 88 x 11 x 2 m.

gramme was dependent upon the weir, which was the more important object as its construction was delayed by the navigation which had to be maintained during the whole of the construction period.

Construction began with the first opening on the right abutment which was carried out in an open site behind cofferdams, the other piers were founded on compressed-air caissons. The weir sills were all built behind cofferdams spanning the openings between piers. The construction was facilitated by two temporary bridges erected across the Rhine on temporary square-faced concrete piers, each of these marking the upstream and downstream limits of the final piers. A heavy steel movable gantry spanning the two bridges was used to suspend the caissons between the upstream and downstream temporary piers. As the sides of the caissons were extended upwards in reinforced concrete, the gantry was relieved from the extra weight by gradually lowering the whole caisson into the water. The whole weir and the power house are founded on sandstone and alluvium gravel; only the lock came on chalk containing gypsum, which being harmful to concrete creates a special problem. On the left river bank altogether 1,800,000 cu. m. of gravel and 200,000 cu. m. of sandstone were excavated, a small proportion of which was used as aggregate for the concrete (250,000 cu. m. of concrete for the whole structure), the rest being transported upstream of the harbour to raise the river bank for future extension of the harbour.

The Birsfelden project was designed by the Consulting Engineers A. Aegerter and Dr. O. Bosshardt, to whom we are indebted for most of the information contained in this paper and for the illustrations.

Book Reviews

Wasserkraftmaschinen. Eine Einführung in Wesen, Bau und Berechnung von Wasserkraftmaschinen und Wasserkraftanlagen. (Water-power Machines, an introduction to the nature, construction and calculation of water-power machines and plants), by Dipl. Ing. L. Quantz. 10th enlarged and revised edition. Published by Springer-Verlag, Berlin-Göttingen-Heidelberg, 1954, 6 in. by 9 in., VIII-159 pp., 226 ff., 2 tables. Price in Germany, paper covers, D.M. 11·40.

As indicated by its sub-title, this work is intended as an elementary text-book, and has, as such, acquired quite a reputation in Germany since it was first published in 1907. It now appears in a new edition which covers most of the latest improvements achieved in the construction of water turbines. The first two chapters give general data on water-power plants and machines, in addition to a review of the evolution of turbine design since Fourneyron, Henschel and Jonval. Chapter II contains a brief survey of the theoretical aspects, such as the motion of water in turbines, hydrodynamics, behaviour of turbines under variable conditions, specific speeds, etc. Chapters III to V are each devoted to one of the three main types of turbines met with in practice. The chapter on Francis turbines (47 pp.), with its numerous calculation examples, deserves special credit and should prove particularly useful to students. On the other hand, the two chapters on propeller (Kaplan) and Pelton turbines are unexpectedly short, covering no more than 19 and 13 pp. respectively. Tubular turbines are briefly discussed.

Chapter VI, which concludes the work, deals with the standardisation of water turbines, and contains suggestions well worth considering. Two appended tables (with graphs) are included, one for the determination of the most suitable turbine design and the corresponding capacity and speed for given heads and discharges, and the other for calculating the most advantageous diameter of runner under given conditions. Line illustrations are generally excellent, though some of them appear somewhat obsolete; the same applies to photographic illustrations which, in part, seem to have been reproduced from earlier publications. Print and paper leave nothing to be desired.

Calcul des Conduites d'Eau avec Cheminées d'Equilibre. (Calculation of Pipes provided with Surge Tanks). By A. Ghizzetti. Published by l'Institut National pour les Applications du Calcul (Rome). Translated from Italian to French by Angelo Poli. 1953. Gauthier-Villars, 55, Quai des Grands-Augustins, Paris.

This volume is the first of a new series of textbooks and monographs to be published under the editorship of M. Louis Couffignal on technical calculations. It was originally written in Italian by Professor A. Ghizzetti, of the Italian National Institute for Practical Calculations, Rome, and translated into French.

The author starts from the well-known basic equations for surges (dynamic equations and equation of continuity of masses) which lead to a non-linear equation of the second order. Instead of writing this equation as a function of the velocity in the pressure tunnel, or of the vertical velocity in the surge tank, as is done usually, the equation is written with the surge height Z as variable quantity. The inflection points and the maximum and minimum of the integral curve are analysed, and tables of functions are given for the calculation of these particular points.

The whole calculation is simplified by introducing dimensionless values, similar to those introduced by Calame and Gaden and by Vogt.

The method allows for rapid calculation of the successive maxima and minima of the surges (a problem the solution of which by direct calculation is tedious). The method is developed for the case of instantaneous partial or total closing of the gates, and for a cylindrical surge tank. Later on the method is extended to a surge tank with varying horizontal section. All these cases have years ago been solved by the well-known methods of Calame and Gaden.

The important cases of a turbine provided with a governor and of linear closing or opening are not considered in this publication.

Petter Oil Engines. A new booklet describes the entire range of petrol, diesel and TVO engines manufactured by Petters Limited of Staines. These well-known engines are widely used on civil-engineering works for driving air compressors, pumps and generating sets, and the booklet forms a convenient work of reference for the sizes and types available.

Present Trends in the Design of Pressure Tunnels and Shafts

An abstract is given of a paper presented to the Institution of Civil Engineers by Charles Jaeger, Dr ès Sc. Techn., the text having been modified in places in the light of contributions made to the discussion.

PART ONE

MANY years ago, engineers were vastly impressed by the bold design of the great trans-alpine tunnels (St. Gothard and Simplon). Data were given at that time about the internal rock pressure and the creep of the rock; examples of tunnel linings being squashed by the rock pressure were mentioned. These were interpreted by the theory of A. Heim (Zurich University) according to which, at great depths, its own weight causes the rock to behave as a plastic material or even as a liquid: the pressure around any point is the same in all directions and equal to the hydrostatic pressure, the density being that of the rock.

When, some years later, engineers were requested to design the first large hydro-power pressure galleries in rock, or to submit proposals for the first steel-lined shafts, they were strongly influenced by the unfortunate history of the Ritom tunnel,[22] which had been damaged and dangerously fissured by internal water pressure.

Designers tried to develop theories explaining the strain and stress distribution in rock as a special case of the theory of thick cylinders. Not only did the two theories of rock deformation seem to have no connection whatsoever, but they seemed to contradict each other. Another theory was being developed, assuming that an empty tunnel in the rock is subjected to a vertical field of parallel forces, so contradicting the theory of Heim and obscuring the issue. However, it can now be appreciated that the theory of Heim for empty tunnels and the calculations of the stresses and strains developed around a pressure tunnel are intimately connected. It is probable that future design of underground power stations will depend largely upon the designer's ability to reconcile the two theories and to take advantage of modern prestressing developments.

An interesting example is the design of steel-lined shafts. It is very likely that the first steel-lined pressure shafts were built with little knowledge of the actual stresses developing in the steel lining and in the rock. In the ensuing period, a rule-of-thumb method was used, according to which the stress σ in the lining was supposed to include the whole internal water pressure, neglecting any transmission of load to the rock, and should not exceed the yield stress of the steel plates. The steel lining was therefore calculated by means of the well-known formula for stresses in thin pipes using a high value for σ. This rule is still applied when the rock to be traversed is heterogeneous and somewhat unreliable. The linings of several modern shafts in the Italian Alps were calculated on similar assumptions (with $\sigma = 1{,}600$ to 1,800 kilograms per square cm.). The effect of this rule is to reduce by more than one half the weight of steel to be used for the lining, as compared with the conventional pipeline of identical length, diameter, and pressure conditions.

Table 1 shows the stresses which would develop in the steel lining of actual shafts on the basis of this

TABLE 1.—CHARACTERISTIC DATA ON SOME GREAT STEEL-LINED PRESSURE SHAFTS

Power station	Country	Gross static head:		Internal pipe diameter:		Steel thickness:		Stresses * in steel plate:	
		m.	ft.	m.	ft.	mm.	in.	kg./sq. cm.	lb./sq. in.
Handeck I	Switzerland	550	1,800	2·10	6·89	20	0·79	2,885	41,000
Innertkirchen	,,	672·3	2,200	2·40	7·87	20	0·79	4,030	57,400
Innertkirchen test tunnel	,,	1,500	4,920	2·20	7·22	10	0·39	16,500	234,000
Handeck II	,,	463	1,520	2·15	7·05	12	0·47	4,150	59,000
Oberaar shaft	,,	531	1,740	1·65	5·41	13	0·51	3,380	48,000
Oberaar free pipe	,,	532·5	1,743	1·40	4·59	28	1·10	1,330	18,900
Wassen	,,	279·2	915	2·40	7·87	11	0·43	3,050	43,100
Gondo	,,	490	1,605	1·6	5·25	17	0·67	2,300	32,700
Verbano	,,	297	974	2·8	9·19	26	1·01	1,600	22,800
Piage-de-Vizille	France	142·5	466	3·5	11·48	11	0·43	2,270	32,200
Isère-Arc	,,	152	500	3·7	12·14	16	0·63	1,750	24,900
Kemano	Canada	790	2,597	3·35	10·99	49·3	1·95	2,670	37,900
Pfaffensprung (projected)	Switzerland	818·5	2,680	2·50	8·20	34	1·34	3,000	42,500
Abjora	Norway	442	1,450	2·30	7·55	22	0·87	2,310	32,900
Lyse	,,	630	2,067	2·0	6·56	18	0·71	3,500	49,700
Tafjord	,,	396	1,300	1·75	5·74	15	0·59	2,410	34,300
Rössaga	,,	245	804	2·8	9·19	23	0·91	1,500	21,350

* At the bottom of the shaft, assuming that no load is transmitted to the rock.

simplified assumption.

In the years 1925 to 1933, theories [20, 21, 23, 49, 50] were developed which took into account the relative elasticities of the steel lining, the concrete, and the rock, and considered the partial transmission of the water pressure from the steel lining to the concrete and the rock. The principle of this calculation is easily understood[49, 50]:—

The water pressure p inside the conduit (Fig. 1) produces an expansion of the steel lining which is

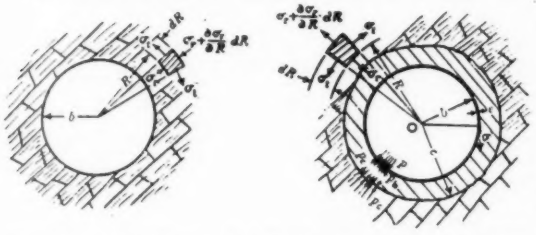


Fig. 1 (a, left). Stress distribution in rock around an unlined pressure tunnel, and (b, right) stress distribution between steel lining, concrete and rock in steel lined pressure galleries

partially restrained by the concrete and the rock. If p_b and p_c denote the pressures transmitted in a radial direction from the steel to the concrete and from the concrete to the rock respectively, then tangential stress σ in the steel lining is $\sigma=(p-p_b)b/e$, where b denotes the radius of the steel lining and e its thickness. If E denotes the modulus of elasticity, then the elastic radial displacement of the lining is $\Delta b=\sigma b/E$, which must be equal to the radial displacement of the concrete. A second system of equations is written for the radial displacements of the concrete and of the rock, which must also be equal. The two systems of equations yield the values of p_b and p_c, and the stresses σ_r (radial) and σ_t (circumferential or tangential) at a distance R from the centre O.

The calculation shows that for small water pressures p (up to 5 kg. per square cm.) and small diameters D (2·5 to 3·5 metres) the tensile stress σ in the steel lining is low, even when the steel thickness e is small.

Many measurements of stresses in steel linings have confirmed this conclusion and so proved that the bulk of the load is taken by the rock, as foreseen by the theory. If the pressure or the diameter increase, the proportion of the load taken by the steel lining also increases.

Another conclusion is that circumferential tensile stresses σ_t develop in the rock. The calculation also shows the great importance of temperature stresses and of any gap existing between steel lining and concrete lining before the tunnel or shaft is put under pressure.

Because of these tensile stresses in the rock, it was thought that there would be a limit to the dimensions of steel-lined pressure shafts. Theories regarding what would happen if radial fissures were to develop in the rock were propounded but were not convincing.

In contradiction to the theory, tensile stresses developing in the rock seem to have no effect on the stability of the lining; rock which should be fissured remains solid and doubts may therefore arise with regard to the validity of the theory.

Up to the present no failure of steel linings arising from the development of rock fissures has been reported. But failures of the lining have been caused by external water pressure, by grossly defective design, or by lack of homogeneity of the concrete filling between the steel lining and the rock.

A direct proof that the strained rock behaves quite differently from what could be expected from the theory is given by the behaviour of three Italian pressure shafts designed as early as 1925/1926 by Marinoni, the characteristic data of which are given in Table 2. These three shafts are lined with corrugated steel plates, the thickness of which is less than 4 mm.; these cover an ordinary concrete lining. The stations have been in operation for 25 years and the accidents which have occurred to the shafts have all been caused by outside pressure and were quickly repaired. This experience shows that in some cases the whole hydrostatic load can be transmitted to the rock, the steel lining serving only as a watertight sheath.

The theoretical values calculated for σ_t in Table 2 are higher than those which would normally be assumed for a homogeneous rock and much higher than for a fissured rock.

Clearly, therefore, the ability of rock to withstand the tensile stresses σ_t is far greater than was expected. As a result new theories had to be evolved on the basis of extensive research and measurements, and these are of great importance for the technique and design of pressure shafts.

Most of this new research work assumes, either explicitly or implicitly, the validity of the old theory of Heim concerning the internal compression stresses already present in rock prior to any excavation. This rock pressure, which at some greater depth may be approximately the same in any direction at any particular point, is, in the mind of the designer, equivalent to a natural prestressing of the rock. This natural prestressing may more than compensate the tensile stresses induced by the water pressure in the tunnel.

A tunnel or shaft will be safe if the resultant stresses at any point of the steel lining or at any point of the rock, including the effect of prestressing, are well inside the known permissible values.

These introductory remarks on the theories of rock behaviour will make it easier to present the more detailed analysis of the complicated conditions prevailing in lined and unlined pressure tunnels and shafts.

The next logical steps in the discussion of this subject are consideration of the constructional materials

TABLE 2.—SHAFTS DESIGNED BY MARINONI

	Head:		Diameter of shaft:		Rock	Theoretical value of σ_t in rock:	
	m.	ft.	m.	ft.		kg./sq. cm.	lb./sq. in.
Pallanzeno .	527	1,730	1·80 to 1·50	6 to 5	Gneiss . .	−52·7	− 750
Rovesca .	720	2,360	1·20	4	Schists and gneiss .	−72·0	−1,025
Mese . .	758	2,490	1·80	6	Gneiss (not compact) .	−75·8	−1,075

available and an analysis of the behaviour of concrete, steel, and rock. Rock is by far the most important material and very little is yet known about its behaviour *in situ*.

Concrete for Tunnel Linings

Only vibrated concrete is now used for filling between the steel lining and the rock. This filling must be homogeneous, without voids, and as impervious as it can be made.

The technique of vibrating the concrete is the same as that which has been developed for dams. In order to be sure that the filling is as perfect as possible, there must be a limit to the length of the pipe sections used for lining, and special care has to be taken with the anchorage. Table 3 gives some examples of the thickness of the concrete filling, the pipe lengths used, and types of anchorage.

Extensive use has been made of the concrete-pump on several shafts. The new concreting technique used at Kemano is well worth mentioning. After considerable study, the designers determined that better control would be afforded by using the Prepakt method. Point-to-point contact of the rock and aggregates minimises the contraction of the concrete.

The aggregates, ranging from 1 to 3 inch size, are chuted into the space behind the steel liner (a space with an average width of 2 feet) through a 10-inch diameter pipe, which is kept full at all times to minimise breakage of the stones.

The grout mixture includes cement, fine sand, an air-entrainment mixture, and a wetting agent. It is pumped into the aggregates through grout nipples placed in the pipe during fabrication. Four holes are arranged around a horizontal plane and the grout is pumped into these holes until it emerges from holes at a level 7 feet higher. Then the grouting equipment, which is mounted on three interconnected platforms, is moved up.

TABLE 3.—THICKNESS OF CONCRETE PACKING, LENGTH OF PIPES, AND DESCRIPTION OF ANCHORAGES FROM STEEL TO CONCRETE

Pressure shaft	Thickness of concrete:		Length of pipes:		Anchorages,
	m.	in.	m.	ft.	
Innertkirchen .	?	?	10–12*	33–39	None
Handeck II† .	0·06	2·33	?	?	None
Oberaar† . .	0·10–0·13	4–5	10	33	60/20 to 80/25-mm. angles. Six on a length of 10 m.
Verbano . .	0·15	6	10	33	
Gondo . .	0·2–0·35	8–14	8–12‡	26–39	None
Isère-Arc . .	0·40	16	3	10	100/100/10 to 120/120/12-mm. angles. Two hoops per 3 m.
Kemano§ . .	0·6	24	2 × 4·3 or 2 × 8·6 welded	2 × 14 or 2 × 28 together	Prepakt concrete, 1–3 in. placed by 10 in. pipe
Proposed by Mr. Talobre for new designs . .	0·25	10	—	—	Hedgehog hair

* Owing to insufficient rock overburden, the distribution pipes of Innertkirchen are enclosed in a second pipe, which has a wall thickness of 20 mm., the clearance between the pipes being 20–40 cm.

† Concreting pipes were used. (The concrete for Gondo was a rich mix, with 275 kilograms of Portland cement and 50 kilograms of pozzolanic cement per cubic metre of concrete. An air entraining agent was also added.)

‡ On the horizontal section the pipe length was restricted to 8 metres because of limitations on the weight sections of pipe.

§ "Prepakt" concrete, aggregates 1–3 inches, placed by 10-inch pipe. Two lengths of pipe were welded together and the assembled sections then brought to the inclined shaft.

At the Maggia pressure shaft the wet concrete, containing an excess of water and sand, slides down small inclined channels a distance of about 500 metres. The excess of sand avoids segregation of the concrete, which, when set, is watertight but has a low crushing strength.

It will be seen from Table 3 that in some cases the steel linings are reinforced by steel hoops and anchored in the concrete. The designers (OFINCO, Geneva) of the Gondo power station definitely rejected any anchorage because of the danger of producing voids in the concrete.

During the concreting process at Gondo, the steel lining was reinforced by internal strutting. Concreting of the section of the shaft not lined with steel was carried out by means of a movable braced shutter, 4 to 5 metres long, moving at 30 to 50 centimetres per hour. Concreting was continuous by day and night, including week-ends.

Other designers insist that this anchorage is necessary to prevent buckling, when the pipe is emptied, owing to the external water pressure arising from seepage through the rock. Some designers have produced interesting proposals to avoid the formation of voids at the anchorage hoops and to facilitate the filling of them when grouting between the steel lining and the concrete.

It has recently been proposed to use expanding cement for the concrete packing of the steel pipes.

Steel for Tunnel Linings

The steel most commonly used for the lining is ordinary steel, as can be seen from Table 4.

At Oberaar the lower part of the conduit passes underneath the Grimsel reservoir basin. Since the overburden is comparatively shallow, the rock cannot withstand the high bursting pressures. For this section the lined shaft is replaced by a steel pipe lying free in a tunnel so that no stress can be transmitted to the rock. For this stretch of free pipe the steel used is a Cu.–Cr. steel with an ultimate strength of 50 to 60 kg. per square mm. (32 to 39 tons per square inch). The diameter is 1·40 metre and the steel thickness 28 mm.; therefore, the stress in the steel pipe for hydrostatic load (disregarding the pressure gradient) is 1,330 kg. per square cm. (see Table 1). All pipe sections were tested at works. This pipe was both welded and stress-relieved at site. A special coupling has been designed to connect this special steel pipe to the ordinary steel plates used for the lining of the shaft. This is the first application of special alloy steel for pressure shafts and it may lead to further developments.*

The welding of transverse joints inside a shaft is a difficult operation. Radiographic checking of the welds is now becoming a routine technique at several sites. Isotopes of irridium (192) and cobalt (60)

* See footnote on page 57.

TABLE 4.—GRADES OF STEEL USED FOR LINED SHAFTS: COMPARISON WITH BRITISH PRACTICE FOR PIPELINE DESIGN

	Ultimate strength :	
	kg./sq. cm.	tons/sq. in.
Steel commonly recommended in Great Britain for pipelines:		
B.S. 15, structural steel	4,300–5,100	28–33
B.S. 14, marine boiler plate	4,300–5,000	28–32
B.S. 534, water and gas lap-welded pipes	3,700–4,300	24–28
Malgovert: prestressed pipe	5,600	36·5
Glockner-Kaprunwerke	4,400–5,800	28·5–37·5*
Innertkirchen	4,300–4,800	28–31
Oberaar :		
Ordinary lining of S.M.I. quality	?	?
Cr.-Cu.-steel for free pipe section	5,000–6,000	32–39
Gondo:		
Ordinary steel A.35	3,500–4,200	23–27
Special steel A.42	4,200–5,000	27–32
Kemano:		
$e < 1$ inch, A.S.T.M. A.285 fire-box quality grade B		
$e < 1$ inch, A.S.T.M. A.201 fire-box quality grade A		

* Steel containing 0·02 to 0·08 per cent. aluminium $\sigma_{elas} = 0{\cdot}65$ to $0{\cdot}68 \times \sigma_{ult}$.

were used at Oberhasli; cobalt was used at Kemano.

Tables 1 and 5 summarise the data available for some of the large steel-lined shafts now in existence and these can be compared with the figures given in Table 4. Table 3 gives the lengths of the pipe sections used at different places.

Crushing Strength of Rock

Natural rock is the most important material in steel-lined pressure shafts since it carries most of the pressure loading. Being well aware of visible or assumed faults in rock, civil engineers show some hesitation in permitting it to be subjected to a high proportion of the water-pressure stresses.

The rock properties which are important in the design of steel-lined shafts are its crushing strength and tensile strength, its modulus of elasticity, and its imperviousness. But, above all, the natural prestressing of the rock provides the key to its behaviour.

The crushing strength of sound rock as ascertained from unfissured samples is often very high. Values of up to 2,000 kg. per square cm. have been obtained from Urseren granite (Swiss Alps). The crushing strength is greatly increased if a transverse strain is applied. Fig. 2 (after Talobre[25,26,27]) shows the Mohr circles corresponding to a rock sample with a crushing strength σ_{max} of 300 kg. per square cm.; when a transverse stress component $\sigma = 100$ kg. per square cm. is applied, the crushing strength rises to 600 kg. per square cm., although the same rock is said to have a tensile strength of 60 kg. per square cm. It is possible to trace, for varying conditions of transverse strain, the envelope to all the Mohr circles; it is a curve showing the final strength of the assumed homogeneous and unfissured rock.

If the rock is fissured, then its crushing strength is not decreased so long as the angle $\frac{1}{2}\pi - \alpha$ between the direction of the compression stress σ_1 and the direction of the fissures is such that $\alpha \leqq \phi$, where ϕ is the angle of friction between rock and rock. Obviously, $\alpha = \phi$ is a limit at which the conditions of equilibrium of the sample will change. If $\alpha > \phi$ and if no strain component acts in a direction perpendicular to σ_1, the sample shows no resistance to the compression force.

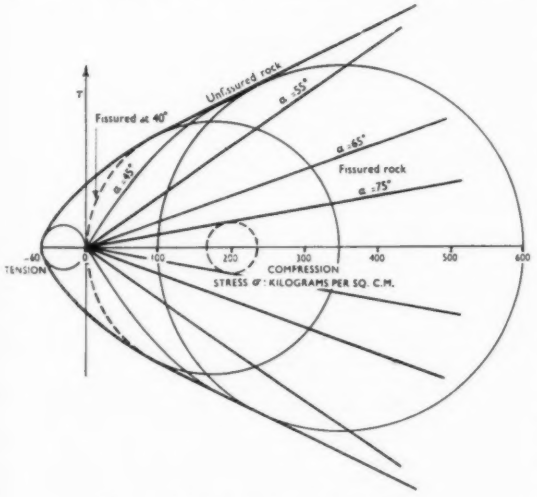


Fig. 2. Mohr circles for rock

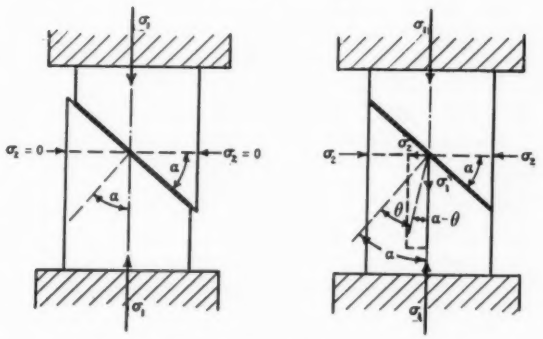


Fig. 3. Lateral stress in fissured rock

When a stress σ_2 perpendicular to σ_1 is introduced, then, according to Fig. 3, the angle θ of the stress ρ resulting from σ_1 and σ_2 must be $\theta \leqq \phi$. Then $\sigma_2 = \sigma_1 \tan(\alpha - \theta)$ and therefore $\sigma_2 \geqq \sigma_1 \tan(\alpha - \phi)$.

In Fig. 2 it is assumed that $\phi = 40$ degrees and the curves for the limiting crushing strength for $\alpha = 45$, 55, 65, and 75 degrees respectively are traced using Coulomb's law for combined shear and friction.

When an unlined pressure gallery is submitted to an internal pressure p, the resulting circumferential stress σ_t developed anywhere in the rock at a distance R is always equal to $\sigma_t = -\sigma_r$, where σ_r is a radial compression stress whilst σ_t is a tensile stress. $\sigma_r = -\sigma_t = p$ for $R = a$ = radius of the gallery; σ_r and σ_t are principal stresses. In Fig. 6 the line OAA′ is assumed normal to the direction of the strata, which has an inclination $\alpha = 0$. If the point B′ moves from A towards infinity, the angle α increases and—when neglecting the overburden—the resultant stress has a tendency to open the fissures as soon as $\alpha = 45$ degrees,

the tendency being obviously to a maximum along D′D′.

If the rock is of poor quality or if the overburden of the rock above the unlined tunnel is not sufficient, cracks may occur along D′D′ and water may seep through the fissures. Cases of severe damage are known, especially where landslides have been caused by the seeping water.[22, 25, 49, 62]

The "overburden-action" will keep the fissures closed and check the detrimental effect of the tensile stresses irrespective of the angle β of the strata to the horizontal. The explanation of this action requires additional investigation and measurements, the results of which have only recently become available.

Elasticity of Rock

In 1923 the pressure tunnel of Ritom in Switzerland was severely damaged by cracking, attributed to yielding of the rock.[22] Measurements were made in the tunnel of Amsteg, in order to determine the modulus of elasticity of the rock *in situ*. These measurements were probably the first to be published on this subject. Later on, similar tests were repeated in many tunnels[31] (Lucendro, Malgovert, Rossens, Italian tunnels, etc.), sometimes in conjunction with the construction of large dams, but mainly in order to get first-hand information on the behaviour of pressure tunnels. The results obtained showed that the modulus of elasticity E depends not only on the rock type and quality but also on the direction of the force T, E being smaller in the direction perpendicular to the strata than in the direction parallel to the strata. The measured values in both directions are:—

	Normal to strata: kg. per sq. cm.	Parallel to strata: kg. per sq. cm.
Rossens sandstone . .	22,000	50,000
Urseren granite . .	250,000	400,000
Urseren schists . . .	80,000	280,000

There is an interesting comparison between these figures and the value of E measured at Chimbote in

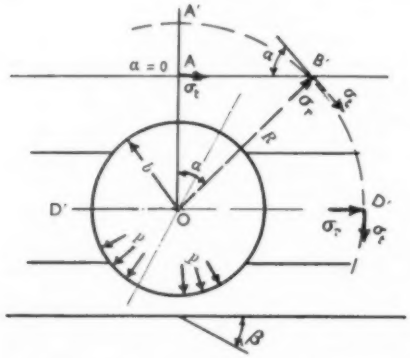


Fig. 4. Pressure gallery in inclined fissured strata

Peru, which is 350,000 kg. per square cm. More recently P. Bernard measured 66,000 to 190,000 kg. per square cm. for schists, 250,000 to 455,000 for quartzites, and only 2,000 to 26,000 for sandstones.

Apart from the very considerable differences between the values for a hard homogeneous rock like granite (Urseren) and a good compact sandstone (Rossens), the importance of the direction of measurement relative to the strata is obvious. It can be assumed that the lower value of E (measured normal to the strata) is attributable to the plastic deformation of the fissures, which first have to be closed before elastic deformation becomes possible.

If a cycle of loading, ranging from zero to maximum, is repeated several times, then the plastic deformation can be separated from the elastic rock

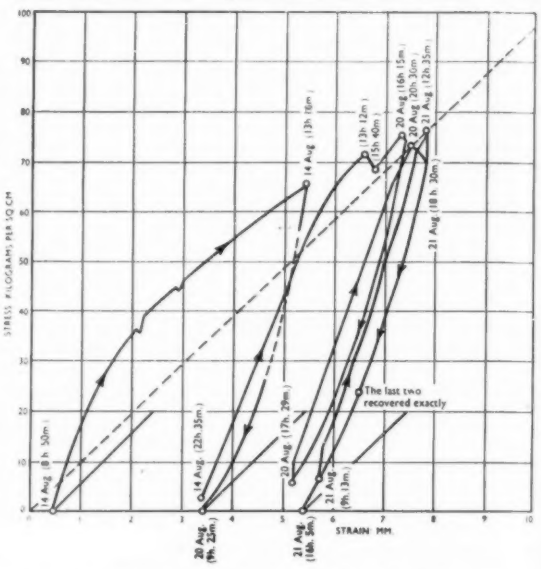


Fig. 5. Plastic and elastic rock deformation in Malgovert tunnel (after Talobre)

deformation, as shown by Fig. 5. This diagram was derived by Talobre from tests inside the Malgovert tunnel,[25] where the pressure was raised to 60 to 75 kg. per square cm.

After five applications of the loading cycle in a period of 7 days, the rock was behaving elastically. The modulus of elasticity was then nearly equal to the so-called "dynamic modulus" as obtained by laboratory tests on a dense rock sample using acoustic methods. Similar diagrams were published by Oberti[43, 44, 45] as early as 1944, in a research paper of great importance, in which much pioneering research work was first made available to engineers.

The experiments by Gicot at Rossens yielded similar results.[31, 36] Frey-Bär carried out measurements[31, 38] on several stretches of the Lucendro tunnel, in order to test different types of gneiss. The hydrostatic pressure used for the tests was about 10 kg. per square cm. Both rough and lined sections were tested. The method of testing involved measuring the length variations of two mutually perpendicular diameters, and was similar to the method used years earlier in the Amsteg tunnel. The measuring apparatus was of the Gerber type built by Huggenberger.

It was found that, in so-called Para-gneiss, the overall modulus E', including plastic deformations, was $E' = 120{,}000$ kg. per square cm.; whereas the elastic modulus was $E = 390{,}000$ kg. per square cm.

A concrete-lined section of the tunnel with steel reinforcement bars proved to have a value of $E' = 150{,}000$ to $230{,}000$ kg. per square cm., 20 to 28 per cent. of the strain being plastic deformation of hysteresis type.

The technique used by Oberti[31, 37] in badly fissured

limestone was very similar to that used by Frey-Bär. He observed a high proportion of plastic deformation at the beginning of the tests and found that E was decreasing with increasing pressures. Grouting of the fissured rock definitely improved the modulus E. He found that, for a pressure p (Fig. 6) between 8 and 15 kg. per square cm., E varied from 35,000 to 52,000 kg. per square cm. before and after grouting.

Quite independently, Bernard[17] made further similar tests, measuring the strains in tunnel linings during and after injection and determining the stresses before and after grouting. His tests showed that the lining contracts during the grouting process (to a great extent permanently); the stresses under loading are reduced and are equalised in the various planes; the deformations measured in the earlier tests were almost eliminated by the grouting.

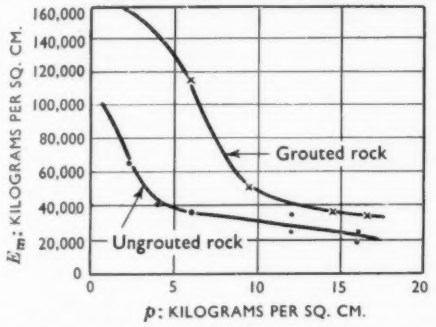


Fig. 6. Effect of grouting on the modulus of elasticity of fissured rock (after Oberti)

Tests in schists were carried out with grouting pressures of 30 kg. per square cm. (absorbing about 175 kg. of cement per square metre) with the result that the modulus of elasticity was improved from 40,000 to 100,000 or even 236,000 kg. per square cm. It is likely that grouting is efficient in hard fissured rock, where the fissures are closed by the grout. Tests in sandstone and marl were attended by little success.

It should be recognised that the results of this last series of tests are of the greatest importance, and it is highly desirable that they should be repeated in all types of rock. The effect of pressure grouting in decreasing the ratio E/E' should be further investigated.

The modulus of elasticity of rock and its compressibility can be obtained by measuring the yield of the rock when subjected to a concentrated load, as at Rossens.[36] Alternatively, sections of tunnel have been sealed off and filled with water under controlled pressure. The variation in the tunnel diameter or circumference was then recorded as the pressure on the water was increased. The method has been used at Amsteg, Innertkirchen, Handeck, Rossens, Lucendro, and for several French, Austrian, and Italian tunnels.

Acoustic or seismic methods are also used, the rate of transmission of sound vibrations being related to the degree of fissuring of the rock—the more compact a rock, the higher the wave velocity. This method enabled fissured rock to be detected and avoided during the construction of the inclined shaft of Carla hydro-electric power station near Toulouse.

Recently Gigli in Italy and Remeniéras of Electricité de France have suggested[47, 48] using the waterhammer pressure waves for measuring the overall E value in a tunnel. This method was first proposed in 1934 by the author,[50] when it was established that the wave velocity depended on the respective values of the modulus of elasticity of the rock, the concrete lining, and the steel shell.

Natural Prestressing of the Rock

It is apparent that the rock inside the mountain is stressed by its own weight before any excavation has been made. It is apparent also that this natural stressing or prestressing of the rock increases with the depth. At the beginning of the twentieth century, the Swiss geologist Heim[13] expanded this theory by adding that, at great depths, the stress at any point must be the same in all directions. This is the hypothesis of the so-called "hydrostatic" stress distribution in the rock. A certain amount of backing was given to these theories by the rock behaviour in deep trans-alpine tunnels (Gothard, Simplon, Lötschberg, etc.) and in deep mines.

Measurements have recently been made inside deep tunnels and galleries with the result that Heim's theory has been approximately verified, and other theories must be discarded. However, more specific confirmation will be required if the theory is to become more generally accepted by designers.†

The measurements will have to be made inside the tunnels as they are being blasted out. If it is to be assumed that, before blasting, the rock pressure is uniformly and hydrostatically distributed, the cutting of the tunnel disturbs this pressure distribution. The pressure in a direction normal to the tunnel periphery is released and the corresponding circumferential pressure near the tunnel, measured along a prolonged horizontal diameter, will be increased—perhaps doubled. So the pressure measured along the tunnel wall may not be the actual rock pressure in the undisturbed state.

(To be continued)

* In a recent letter to the author, Dr. A. Kaech expresses the opinion that special steel alloys have a future with steel-lined pressure shafts.

† See comments by Mr. W. H. Ward on the pressure measurements in a gallery driven in London clay, *Proceedings, Institution of Civil Engineers* 1935, Part I.

Reyrolle Equipment. Three new publications issued by A. Reyrolle & Co. Ltd. of Hebburn, Co. Durham, refer respectively to their type L42T vertical isolation switchgear for service up to 33kV (No. 1273); to their type 33PI oil-break switches for up to 33 kV (No. 1279); and to their Control Boards (No. 1248) for various requirements. The pamphlets are excellently produced and illustrated, and the main characteristics are clearly described. The type 33PI oil switches are primarily intended for use with the L42T switchgear but are equally suitable for use on their own.

Blaw Knox Publications. A new series of publications has recently been issued by Blaw Knox Limited. The first of these refers to their heavy-duty motor grader, the new leaflet BK203-BK12 superseding the previous BK153. A new supplement to BK192 and BK203 describes the elevating loader for the Super 12 and BK12 motor graders. Concrete mixers are covered in two new leaflets, one referring to the two-wheeled model (BK189/1) and the other to the four-wheeled model (BK191/1). Publication BK212 is a general circular devoted to the firm's extensive range of earth-moving and constructional equipment.

Fig. 1. Madupatty dam under construction

The Pallivasal Hydro-Electric Development

An account is given by R. Dorai Rajan of a development on the Mudirapuzha river in North Travancore.

THE first power scheme to be developed by the Government of the Travancore-Cochin State is the Pallivasal hydro-electric project, the initial stage of which has been completed and the second stage is now in progress. The headworks of this project are situated in a gorge on the Mudirapuzha river, a tributary of the Periyar in North Travancore hills, at the point where it leaves the township of Munnar. The river in this gorge drains a basin of 92 square miles and having an annual rainfall ranging fro.n 60 to 200 in. It has a total fall of about 2,000 ft. in its passage from Munnar down to Pallivasal, a distance of only 3½ miles. The scheme consists of a river diversion about a mile above the falls, conducting it through an approach channel into a pressure tunnel leading to the edge of a cliff and thence down penstocks to turbines which utilise a resultant gross head of 1,994 ft.

The pioneers of power development to utilise this head were the Kannan Devan Hills Produce Co. Ltd., one of the largest English-managed tea companies, who installed a 1,000 kW plant for their own requirements. When the State Government took up power development on a large scale a diversion weir 130 ft. long was first constructed at Munnar, to retain the water at the desired level of 19 ft. above the bed of

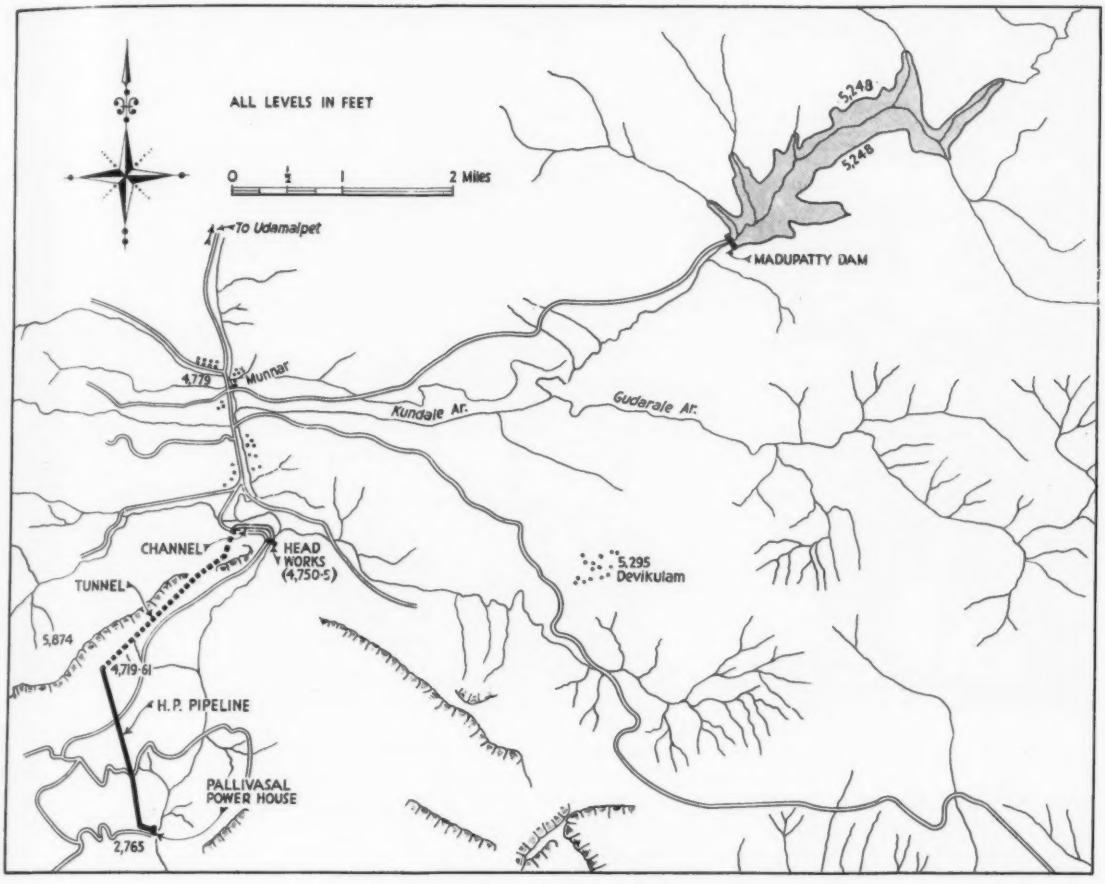


Fig. 2. Locality map of the Pallivasal development

Fig. 3. Kundale dam at Sethuparvatipuram

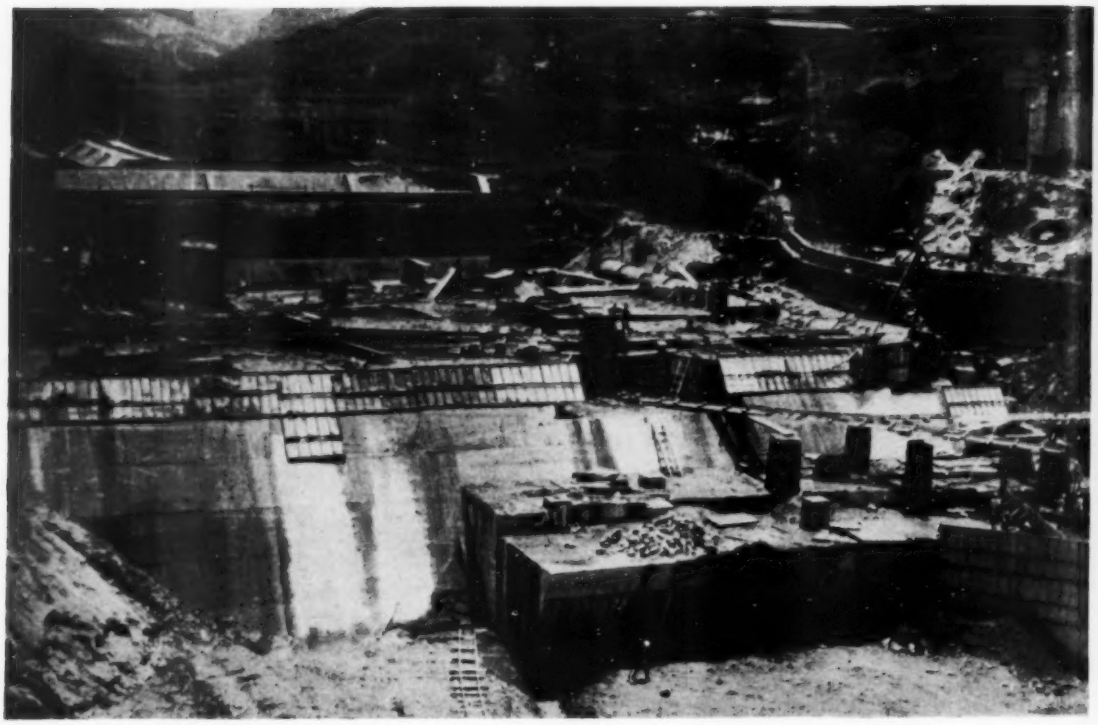


Fig. 4. Madupatty dam, with power-house foundation in the foreground

the river. Three Stoney-pattern gates, 38 ft. wide and 19 ft. high, control the flood discharge, which may reach 45,000 cusecs during the south-west monsoon. A scour sluice is provided for flushing the intake approach. A concrete desilting channel, 1,640 ft. long, with a capacity of 280 cusecs, leads the water from the intake to the tunnel entrance. The main intake to the channel is provided with three gates by Glenfield and Kennedy Ltd., each having a free opening 16 ft. high and 7 ft. wide with trash racks in front.

The main pressure tunnel is 10,235 ft. long and has a fall of 3 in 1,000. The tunnel, like the intake channel, has a capacity of 280 cusecs at a velocity of 6 ft. per sec., the section generally adopted being horseshoe, 9 ft. by 8 ft. A length of about 4,758 ft. of this tunnel is plain concrete lined and 700 ft. provided with reinforcement and gunited. The rest of the length is simply gunited.

The water is then led down two penstocks 30 in./28 in./24 in./22 in. in diameter and 7,635 ft. long and traversing the hill slope at a maximum inclination of 30 deg. Each penstock weighs 1,530 tons. The flow of water is controlled at the valve house by butterfly valves, set to close at overload, and at the turbine end by hydraulically-operated sluice valves supplied by Glenfield and Kennedy Ltd. Both the penstocks were supplied by Ferrum (England) Ltd., each being designed to carry 35 cusecs in normal operation.

The power house, 200 ft. long, 40 ft. wide and 34 ft. high, is located on the right bank of the Mudirapuzha river. The building is a reinforced-concrete structure with walls of hollow concrete blocks. Flanking the main hall on the downstream side is an annexe which accommodates a battery room, an oil filter room, a

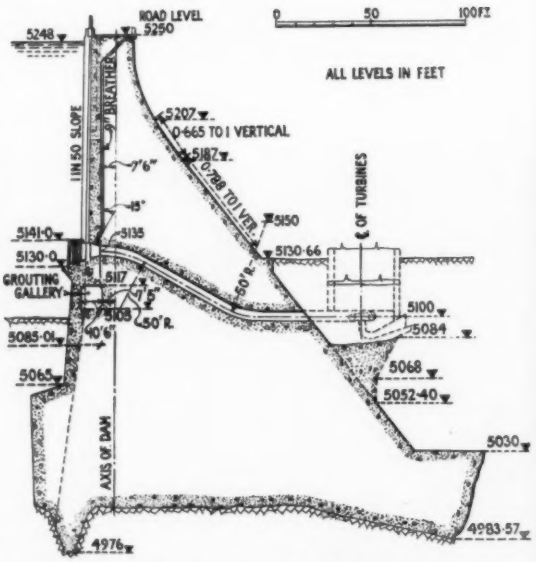


Fig. 5. Section through Madupatty dam

station auxiliary transformer room and, on the ground floor, a room for the 11 kV cellular switchgear and a control room on the first floor.

The three 6,000 h.p. single-nozzle impulse wheels, with a speed of 750 r.p.m., were supplied by Escher Wyss Ltd. of Geneva. They are direct coupled to 4,500 kW 0·9 power factor alternators, installed in the first stage for generation of 9,000 kW of firm

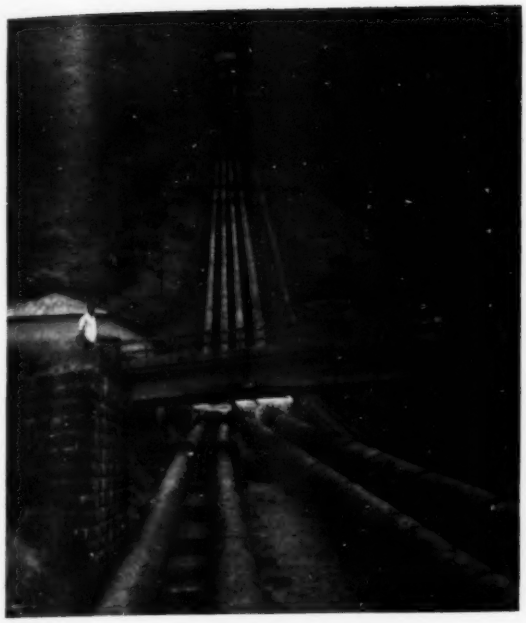


Fig. 6. The Pallivasal penstocks

power, one unit being left as standby. The generators, transformers and switchgear were all supplied by Brown Boveri Ltd.

Owing to the heavy demand for power, the second-stage development, involving three additional generating sets of 7,500 kW each, was undertaken immediately after the first stage was completed. The additional storage required for the second stage has been obtained by regulating the flow of the river with two more reservoirs, one already completed at Sethuparvatipuram and the other at Madupatty Gorge on the Palaar river, a tributary of Mudirapuzha river. The Madupatty dam is well on its way to completion at the time of writing.

The dam at Sethuparvatipuram on the Kundale river, 15 miles east of Munnar, 5,774 ft. above sea level, impounds some 270 million cu. ft. for the Pallivasal station. This dam commands a catchment of 15 sq. miles with an average rainfall of 60 in. It is an arch-type masonry structure aligned to suit the foundation rock. The main dam is 525 ft. long along the vertical face with corewalls 100 ft. on the left flank and 125 ft. on the right flank serving as cut-off walls. It is 86 ft. above the average river-bed level and establishes a hydraulic head of 82 ft. The spillway, 85 ft. wide, is designed to discharge a maximum of 6,500 cusecs through five vents 17 ft. wide and 8 ft. high and fitted with drop-type Glenfield and Kennedy gates. A 10 ft. roadway is provided along the full length of the dam. An outlet pipe of 72½ in. diameter is also fitted in the dam. The volume of masonry amounted to 1·9 million cu. ft.

This storage reservoir maintains a regulated discharge of 125 cusecs and has increased the capacity of Pallivasal station from 9,000 kW to 16,500 kW. Additional storage was required for the second-stage development, and a storage of 1,200 million cu. ft. is the minimum necessary during years of ordinary drought and 2,000 million cu. ft. in exceptionally dry years. To meet this need the Madupatty dam, 5,200 ft. above sea level, is being built, and is claimed to be the first all-concrete dam in India. It is a straight gravity-type structure, 805 ft. long and 280 ft. high above deepest foundation, and controls a catchment of 39 sq. miles. It has a capacity of 1,900 million cu. ft. of water. The spillway is of ogee type designed to discharge a maximum flood of 16,000 cusecs

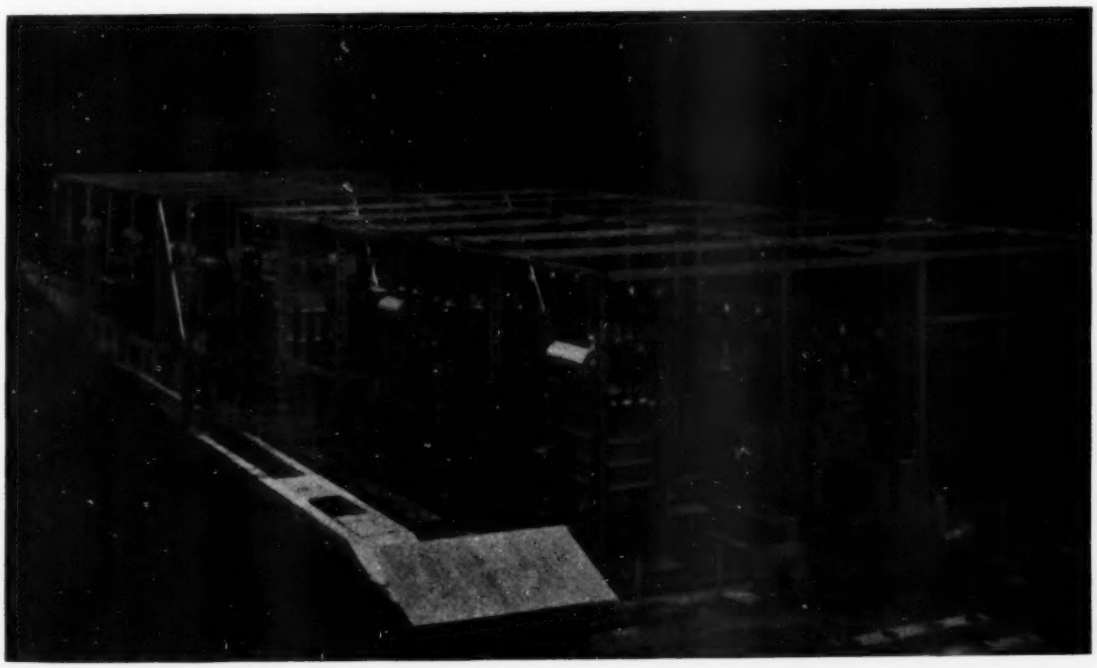


Fig. 7. Outdoor transformer and switching station at Pallivasal

through Glenfield and Kennedy radial gates 22 ft. long and 17 ft. 9 in. deep. Owing to a fault zone and soft patches in the river bed, the conventional central spillway was not placed centrally but was located towards the left bank, the energy of the water being dissipated by the "splitter and step" method. The volume of concrete involved is 5·7 million cu. ft. and the cost Rs 57 lakhs.

A drainage gallery 7 ft. high and 4 ft. wide runs along the length of the dam about 15 ft. from the water face and 15 ft. above the river bed. In addition, a grouting gallery 10 ft. 6 in. high and 7 ft. 5 in. wide is constructed. There are two outlet pipes, 72½ in. diameter, fitted with discharge regulators; one pipe is situated about 4 ft. above the river bed and has a downstream slope of 1 in 50 with trash rack in front, while the other is fitted 34 ft. above the bed level to which it slopes in a curve. Ultimately the latter pipe will feed a low-head power station which will be installed at some future date when the demand for power requires it. Both pipes and disperser valves were supplied by the English Electric Co. Ltd.

A major problem was encountered in consolidating the foundations due to the fault zone in the centre of the river bed. After an initial filling of concrete to a depth of 20 ft., 130 ft. below bed level, mat grouting was carried out at a pressure of 100 lb. per sq. in. to withstand the pressure induced by the dam structure. Percolation will be prevented by a grout curtain of four lines of holes penetrating to 230 ft. below bed level.

Constructional methods on the dam are mostly mechanised. A ten-ton cableway traverses the full length of the dam and has a travelling speed of 1,100 ft. per min. Associated with the cableway is a Ransomes and Rapier batching plant, provided with two mixers of one cu. yard capacity each. In two shifts of 8 hours each it has been possible to place 14,000 cu. ft. of concrete per day with this equipment. The batching plant is the first of its kind to be commissioned in India.

As sufficient sand was not available at the site, a sand-washing plant (already described in the article on the Poringalkuthu Hydro-Electric Scheme*) was installed a few miles upstream of the dam, where there was a convenient deposit of sand underneath the silt. More than 2 million cu. ft. of excellent sand was made available by this process. Had it been necessary to crush rock to replace the quantity of sand required, the excess cost over the washing process would have been about Rs. 10 lakhs.

The flows from the reservoirs at Sethuparvatipuram and Madupatty are led through the natural river to the headworks at Munnar and then through the pressure tunnel, pipeline and power house. In order to feed three more generating units two new penstocks have been laid. One of these is 30 in. in diameter tapering to 22 in. at the power house and the other has diameters of 42 in., 40 in., and 38 in., their discharges being 58 and 116 cusecs respectively. The larger bifurcates at the power house and feeds two machines, while the smaller feeds one machine. The penstocks for the second stage were supplied by the South Durham Steel and Iron Co. Ltd., while the turbines are by Boving and Co. Ltd., coupled to British Thomson-Houston generators. The fourth transformer is supplied by the Westinghouse Electric

(Continued on page 66)

* WATER POWER, Aug. 1953, p. 296.

Fig. 8. Part of the transmission line from Pallivasal

Principles of Tunnel Blasting

The principles underlying placing of holes, construction of drilling and ignition patterns and smooth blasting are discussed. Special attention is devoted to conditions for parallel-cut blasting. A statistical analysis shows how drilling scatter limits the average advance, but a substantial increase in this advance can be achieved, with both wedge-shaped and parallel placing of holes, by precision drilling, large drill holes and exactly-determined sequences of ignition.

By ULF LANGEFORS

PART TWO

Construction of Stoping and Igniting Patterns

The object of the cut is to create free surfaces towards which the rest of the blasting can be carried out. The problem in stoping is to attain just as large advance with the remainder of the round as that effected in the cut, to get satisfactory fragmentation and to get a suitable arrangement and placing of the blasted rock. At the same time one likes also to leave the remaining rock free of cracks and with smooth surfaces.

The advance is dependent upon the breakage conditions at the bottom, and the drilling pattern should therefore in the first place be drawn up there. Departing from the free surface (cross hatched in Fig. 15) the arrangement of holes is indicated with the help of the rectangular principle laterally. The burden increases successively until the practical maximum value is reached. After that one cannot build in quadratic form but with a certain modification (Fig. 15, right). The placing of holes and sequence of ignition according to Fig. 15, left, should be avoided. The sequence of breakage should be specific, and angles of breakage of 90° be aimed at.

Just as much rock as is blasted out with four holes 2, as shown by Fig. 15, left, is blasted out with only two holes 2 as shown by Fig. 15, right, but especially in the pattern on the right, free breakage is not guaranteed for the subsequent holes 3. Because of ignition scatter every individual charge detonates separately and several will probably be wedged in so that there will only be breakage if they are strongly overloaded. There will be a severe strain on the surrounding rock, however, which is torn up along cracks and crevices. This contributes to inferior fragmentation.

The maximum burden and the placing and number of holes are determined by the bottom diameter, the explosive employed and degree of tamping, as well as the factor of the rock and drilling scatter. As already mentioned the effect of these different factors has been made evident in conjunction with calculation of the charge in bench blasting[1]. This makes possible a complete calculation of the number of holes required as a function of the above factors and the area and width of tunnel.

Some such connection for parallel-hole rounds based on the values of the maximum burden indicated in Fig. 1 have been calculated (Fig. 16). It is evident from the curves that a very pronounced reduction in the number of holes is obtained for major tunnel areas

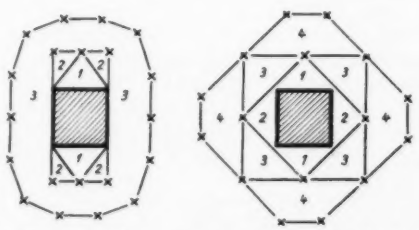


Fig. 15. Construction of stope pattern should be made as in right-hand figure, and not as in the left-hand one

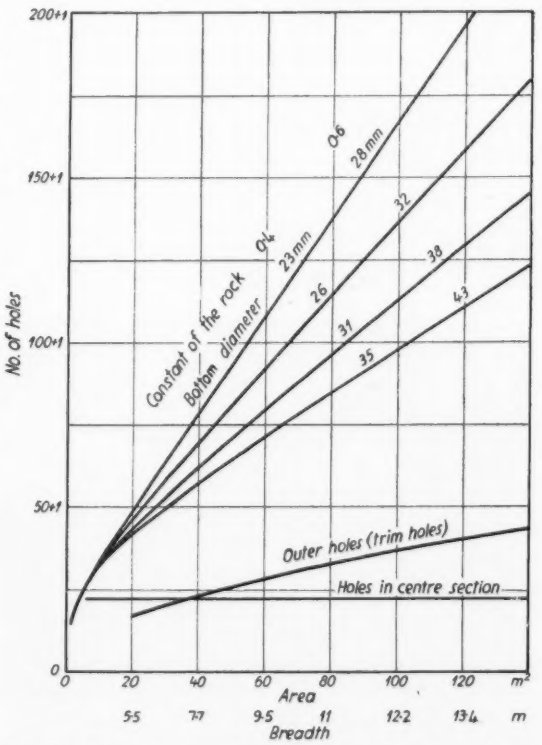


Fig. 16. The least number of holes as function of the tunnel area with parallel-hole rounds for different bottom diameters and for rock constants c=0·4 and c=0·6. The curves include measures for smooth blasting of ceilings and careful blasting of walls

when employing a drill of larger diameter. With a 120 sq. m. area the number of holes can be reduced from about 160 to 110 if we go from 32 mm. to 43 mm. diameter ($c=0{\cdot}6$).

For minor tunnel areas, on the other hand, the difference will be smaller; the corresponding numbers of holes with an area of 30 sq. m. are 58 and 50, which scarcely justifies the employment of a bigger drill. In addition, it is easier in small tunnels to obtain conformity with the given contour when employing small burdens. The curves in Fig. 16 have been calculated on the assumption that this conformity has been possible so that full advantage has been taken of every hole, with the exception, however, of the outer holes which have only been intended for an area of 0·5 sq. m. each.

Examples of some drilling patterns seen at the bottoms of the advances (Fig. 17) for 20 and 55 sq. m. tunnels show that for the former two holes more than the calculated minimum have been required, because the burden for the ceiling holes is greater than desirable and two additional ceiling holes have been inserted. It may be mentioned that a minimum number of holes can actually be secured with a still better solution—a problem that can be left to the reader.

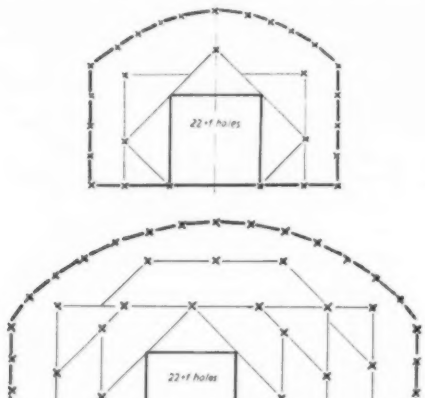


Fig. 17. Drilling patterns for tunnels with blasting towards large centre holes. Top, *area 20 sq. m., number of holes in centre section 22+1, in outer section 19, total 48+1 (46 according to Fig. 16);* middle, *area 55 sq. m., number of holes in centre section 22+1, in outer section 27, total 84+1 (86+1 according to Fig. 16);* bottom, *area 55 sq. m., number of holes in centre section 22+1, in outer section 27, total 69+1 (68+1 according to Fig. 16)*

The two drilling patterns for a 55 sq. m. area illustrate the difference between one stope with small holes and one with large holes (32 and 43 mm. drill respectively in rock with $c=0{\cdot}6$) and show the great simplification of the pattern attained in the latter case. In both cases there are fewer holes (one or two) than in the estimated minimum. This may appear paradoxical, but it is due to the burden at the sides being 1·0 m. instead of 0·75, which is what was intended when calculating Fig. 16. One cannot in general count on obtaining complete conformity. If the condition for the burden of the outer holes is to be maintained there is a possibility that a couple of holes more than the minimum may be obtained, depending on which solution is chosen. In the case of the big-hole stope it is more difficult to adjust the pattern, as has already been indicated.

Placing of the cut influences the throw of the round, the fragmentation and maybe also the number of holes. As the density of the holes is greatest close to the cut the denser placing can be taken advantage of

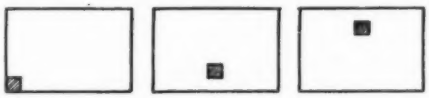


Fig. 18. Different placings of the cut section

if some of the holes are outer ones (Fig. 18, left). This should only apply in minor tunnel sections. In order to secure the best throw and centring of the blasted rock the cut is placed approximately in the middle of the section. Low location gives less throw, larger fragments and closer jamming of the rock. Less explosive can be used by stoping downwards.

In general, however, the problem is the opposite; one wants to avoid employing the free fall. Note that with the given principles for constructing drilling patterns one consistently avoids downward stoping of entire rows of holes on the same interval. If the cut is placed high up in the section a less compact heap of fragments, lower and more extensive with better fragmentation, is obtained.

Smooth Blasting

As a monument to the work done, the walls and ceiling of the tunnel remain long after the blasted portion has been removed and the related technical and economic problem forgotten. In the case of water tunnels, the effects of a badly blasted contour will continually assert themselves through major losses of head; in the case of air-raid shelters and storerooms through increased maintenance costs and decreased strength.

During the past few years, the importance of carrying out smooth blasting of tunnel contours has become more and more evident. Even in this case it is of importance that the blasting is done with free angles of breakage. How the placing of the holes and the sequence of ignition are chosen for the outer section as a whole is shown by Fig. 19. The outer holes should

not be driven to the same depth as the round in general, but should have a lag which is at least as great as the burden. The breakage at the bottom is then obtained without fixation and a smaller concentration of explosive is necessary there, which is made clear by the discussion in connection with Fig. 2.

In co-operation with the Stockholm Harbour Board and the Royal Administration of Fortifications, smooth blasting in vertical drifts and tunnel ceilings has been studied in detail. The most important condition for good results is that the charge and the charge concentration in each hole are small, so that above all heavy overloading and also greater burdens

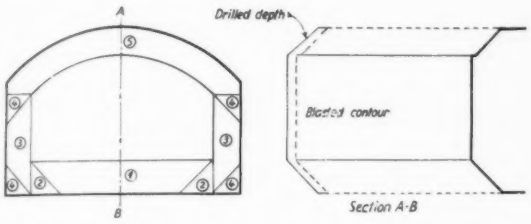


Fig. 19. Blasting of outer sections with least effect on remaining rock

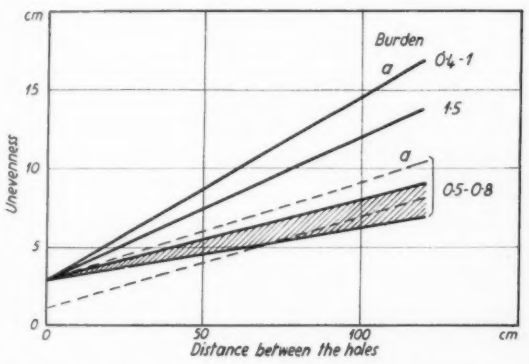


Fig. 20. Average contour of the rock as a function of distances between the holes for different rocks, burden and quantity of the charge.—Working A, *working* B, *curves* a *with heavy supercharging, other curves with correct charging*

are avoided. For the calculation of the necessary charge, reference is made to Ref. 1. A practical and very suitable formation of the charge has been worked out in these trials[6]. The dynamite cartridges are cut lengthwise and placed on half-rods which are turned against the sides of the rock which are to be left intact. With this formation, the connection between the unevenness of the contour of the remaining rock and the distance between the holes is determined (Fig. 20). The values from the two workings *A* and *B* supplement each other, and show how the difference between various rocks affects the picture. Working *B* is in a rather homogeneous and crevice-free rock. Overloading here plays a much smaller part than on the experimental working *A*, which in view of its crevices perhaps better represents a normal rock. Heavy overloading has been used at previous attempts to carry out smooth blasting. As transpires from the sharp inclination of the corresponding curve, it is then

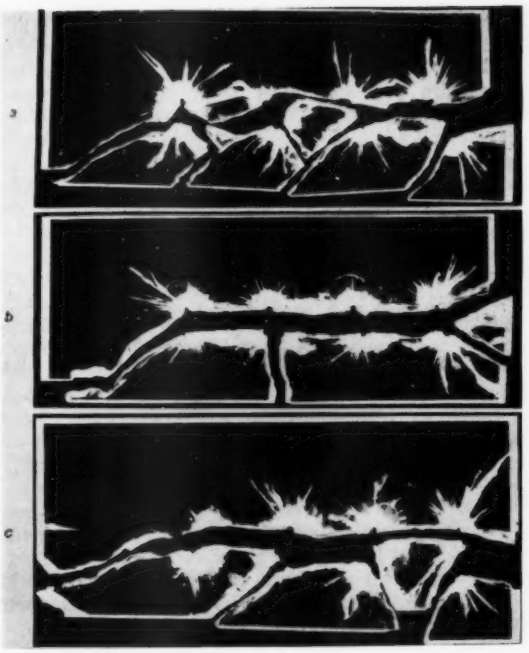


Fig. 21. Blasting of row of holes (experiment in plexiglass); top, *shot by shot;* middle, *instantaneous blasting;* bottom, *short-delay blasting*

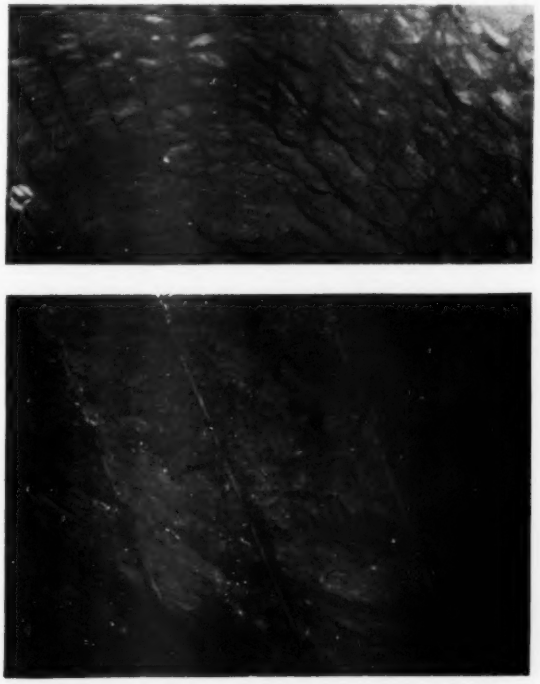


Fig. 22. Result of smooth blasting: top, *cylinder-shaped drift with 1·5 m. radius, distance between holes 0·15—0·20 m., burden 0·4 m. Charge in every second drill hole (The Stockholm Harbour).* Below, *details of ceiling in tunnel with 9 m. span, distance between holes 0·75 m., burden 0·75—1·0 m. (Blåsjöfallet)*

necessary to use close drilling to obtain a good result, at least as far as appearances are concerned. This is presumably the reason for the common and incorrect impression that smooth blasting presupposes close drilling. With correct charges and such rocks as are represented by *A*, this does not apply at all. Distances between holes of even up to 0·75 m. can give an excellent result (unevenness of approximately 6 cm.), at the same time as, for example, a trebling of the number of holes only gives an insignificant improvement of the contour (unevenness 4 cm.). This is partly because in creviced rock, unevenness in the surface cannot be avoided even with seam drilling. In homogeneous rock *B*, as the curve shows, it can be somewhat more profitable with a reduction of the distance between the holes, but then only when extremely high demands are made on the unevenness of the walls. Results, which for the majority of cases of tunnel and drift blasting can be described as highly satisfactory, can be obtained with distances between the holes of 0·5—0·8 m. and burdens of 1·0 m. or less.

The method of ignition also has a great influence on the evenness of the walls. When igniting outer holes one by one, a less satisfactory contour is obtained than when all are exploded simultaneously. Fig. 21 shows results from attempts in plexiglass with ignition shot by shot, instantaneously and with short-delay blasting. It clearly transpires that the most even wall is obtained with the least dispersion of ignition. This has been fully confirmed when checking on a full scale in rock. Of the three alternatives, short-delay blasting is to be preferred if it is a question of bigger charges, as ground vibrations and strains on the surrounding rock then become less. Better fragmentation can also be expected with short-delay blasting.

Conclusion

There are possibilities for a considerable increase in the advance per round in tunnel blasting, but their practical application calls for an extensive knowledge of the decisive factors, and it is not least necessary that experimental resources be pooled and concentrated so that certain "threshold values" can be crossed. There is otherwise the risk that the individual contributions will be of too restricted a nature to give the necessary results.

LITERATURE

[1] Langefors, U. "Laddningsberäkning vid pallsprängning." Nitroglycerin AB, Gyttorp, Sweden, 1953.
[2] Kallin, A. "The Planning of Rock Blasting Operations in Civil Engineering." Manual on Rock Blasting. Atlas Diesel and Sandviken Ironworks, Stockholm, 1952.
[3] Fraenkel, K-H. "Practical Drilling Patterns." Manual on Rock Blasting. Atlas Diesel and Sandviken Ironworks, Stockholm, 1952.
[4] Langefors, U. Slätsprängning. Jernkontorets Annaler, 137, Stockholm, 1953. p. 436.
[5] Steidle, E. "Some Practical and Theoretical Aspects of the 'Burn Cut.'" Joy Manufacturing Co., Pittsburgh, Pa.
[6] "Noggrann utsprängning av bergrumsektioner — speciellt bergrumstak. Redogörelse för viss försöksverksamhet." Rapp. från K. Fortifikationsförvaltningens Befästningsbyrå, Stockholm, Juni 1954.

NOTE. — This article was originally published in Swedish in *Teknisk Tidskrift*, and we are grateful to the editor of that journal for his collaboration and for the use of the illustrations. We are also indebted to the author for supplying the English translation of his article.

From page 62

International Co. Ltd., while the fifth and sixth transformers are by the British Thomson-Houston Co. Ltd.

Two main double-circuit transmission lines erected by the British Insulated Callender's Cable Co. Ltd. convey the power to the Kothamangalam substation, 35 miles away. The transmission and distribution system comprise some 350 route miles of 66 kV lines, 30 route miles of 33 kV lines, 20 route miles of 22 kV lines and 700 route miles of 11 kV lines. Most of the 66 kV lines are supported by lattice-type steel towers and the 11 kV lines by pressure-treated teakwood poles. Until 1950 the transmission and distribution conductors were of copper only, with special spans of cadmium copper; but practically all the transmission and distribution lines erected during the past two years have been of aluminium, in somes cases reinforced with steel.

Side by side with the addition of new generating sets, the waterwheels of the first stage were changed to give a speed of 600 r.p.m., this increasing their rating by 500 kW each. When the second-stage works are completed the installed capacity of the Pallivasal station will be 37,000 kW.

All the six sets generate at 11,000 V, three phase, 50 cycles. The transformer for each generator steps up the voltage to 66,000 V and the synchronising is effected on the secondary side of these transformers. The transformers of the first three units have additional 11 kV star tertiary windings of 1,500 kVA capacity. The generating units and transformers are fully equipped with the necessary protective systems and interlocks on the switchgear to prevent faulty operation. The 66 kV circuit breakers controlling the secondaries of the generating set, transformers as well as of the four outgoing feeders are of 500 MVA rupturing capacity.

The power is distributed over an area of some 9,155 sq. miles, having a population of 7·5 millions. Over 60 per cent. of the power is consumed by certain important industries like aluminium reduction, fertiliser plants, cement, rayons, caustic soda, monazite and titanium production, paper and tea manufacture.

The installed capacity of the various generating stations in the State has been increased to 109,500 kW during 1953-54 by the addition of two new generating stations at Poringalkuthu and Sengulam.

Improved Propeller-type Current Meter. We have received from the firm of A. Ott, Kempten, Bavaria, literature dealing with their improved current meter now marketed under the name of "Ottmeter Mark V," and also known as "Component Meter." In addition to the usual measurements, this meter registers the components of velocity which lie in the direction of its axis, and is particularly useful in connection with angular currents, such as occur in turbine intakes. Within the admissible range of velocities and angles of flow, it is claimed that the component meter operates with an accuracy of $\pm$ 1 per cent. in relation to the cosine rule. A separate pamphlet is devoted to the method of rating applied to the Ott meters, which is fairly similar to that in use at the rating station of the U.S. National Bureau of Standards.

The Oued Agrioun Development

THIS large-scale development, with an average yearly production of 200 M kWh, constitutes one of the major sections of the general plan of utilisation of Algeria's water resources. Inaugurated on June 27, 1954, it comprises the following installations: the dam, power house and tailrace of Iril Emda; the Kerrata and Darguinah works with a dam on the Chabet el Akra, supply tunnel, power house and tailrace; finally, the dam and supply tunnel on the Oued Ahrzerouftis. The water is collected from the heights of Little Kabylia and diverted into a large reservoir, Iril Emda, of 160 million cu. m. capacity. Out of the available aggregate head of 473 m, the underground Darguinah plant operates at no less than 377 m. This plant is fed by the Kerrate balancing reservoir which collects the discharge of the Iril Emda turbines. The Darguinah power house, set up at a depth of 60 m. in an excavation 67 m. long, 20 m. wide and 22 m. high, is divided into three floors: (a) the turbine floor, with two four-nozzle Pelton wheels, a Francis turbine, and the auxiliary generating sets; (b) the alternator floor with two 33,000 kW units for the Oued Agrioun, and a 5,000 kW unit for the Ahrzerouftis; (c) the exciter floor, which serves both as operating hall and erection bay. Two 60-ton travelling cranes, which can be coupled, complete the equipment of the power house. The Oued Agrioun waters, impounded by the Chabet el Akra dam, a 35 m. high arched structure, are led to the surge tank through a "Colcrete" lined pressure tunnel 7·20 sq. m. in cross-section, 8,300 m. in length, with a maximum admissible discharge of 20 cu. m. per second. The surge chamber, 12 m. dia. by 45 m. in height (of which 16 m. are above ground) stands upstream of the gate chamber, from which two penstocks, 1·65 m. dia. and 600 m. long, lead to the turbines. These penstocks are of the Georges Ferrand type and consist of 8 m. long welded sections 10 mm. thick, reinforced by independent steel-cable hoops.

The backbone of the whole development is the group of works at Iril Emda. The rockfill dam itself consists of two sections, linked up by a central spillway block of reinforced concrete which also incorporates, one above the other, the gravel pass and the penstock gallery. The main characteristics of the dam are: a height of 75 m. above the average level of the schist in the river bed; a developed length of crest of about 710 m; maximum head 96 m., minimum head 47 m.; flood discharge 3,500 cu. m. per second. The maximum diverted discharge reaches 30 cu. m. per second, and the installed capacity of 30,000 kVA gives an average yearly production of 38 m kWh. With the exception of the large blocks of calcite or quarry rock on the downstream side, the 6 million tons of rockfill in the dam consists mostly of 300-mm. size stones. The choice of this dam pattern was largely influenced by the proximity of a huge deposit of limestone debris. To cope with flood discharges of as much as 3,500 cu. m. per second, substantial spillways had to be provided; the flood discharge system actually consists of three galleries and of a gravel pass. The spillway block required the placing of no less than 130,000 cu. m. of concrete. Flood discharge is controlled by three sector gates, one of which operates automatically. Each spillway is 10 m. wide, and the layer of water behind the gates reaches a height of 12 m. Erosion in the river bed is restricted to a minimum by an energy dissipator fitted on the downstream end of the spillways.

The upstream works consist of an overflow tower, which also served to keep the water level at E1·505 during the initial impounding stage; a temporary overflow tunnel, the intake tower, and the gate chamber. The latter, built entirely below the bed of the reservoir, is a cylindrical horizontal structure divided into two floors about 20 m. in length; the bottom floor houses the heavy intake equipment, while the top floor serves as a control room and contains the switchboards, as well as the oil tanks of the servomotors of the desilting gates. To utilise the head available to the full, the power house is erected in a shaft sunk at the downstream foot of the dam; this shaft, 18 m. in inside dia., extends in depth to about 45 m. below the floor of the surface buildings and is divided into the following floors—bottom tanks and sumps; pumping plant and pit of the emergency gates; turbines and governing gear; alternators and exciters. Above the exciter floor, there are inspection floors for the cables and ventilation ducts, with a footbridge carrying the cables which connect the generating units on the one hand, and the switchgear and control room on the other. The power-house shaft, as well as the side walls connecting it with the de-silting system, are thoroughly protected from infiltration by strong watertight linings. A special feature of the plant is the provision of an expansion chamber for the temporary storage of the turbine discharge at periods of load variations, especially at starting periods, since the mass of water in the tailrace cannot follow immediately these variations. This expansion chamber is sufficiently distant from the power house to avoid surge in its vicinity, and sufficiently large (10 m. dia. inside) to ensure good stability in the operation of the generating sets. The tailrace tunnel, about 2 km in length and 3.5 m. in diam., is lined with vibrated concrete plates; to ensure smooth working of the generating sets and to restrict head losses to a minimum, the floor lies about 6 m. below the maximum level of the downstream reservoir, so that it can operate at times either as a free-flow conduit or may be under slight pressure.

The care with which this great project of Electricité & Gaz d'Algérie (Electricity and Gas Board of Algeria) was carried into effect is perhaps best illustrated by the fact that a laboratory was set up at Kerata to deal with all the problems likely to arise while construction work was in progress; these included flow measurements in the Oued, solids content of the water, chemical analyses of the watertight facing, compaction of the materials and preparation of the concrete.

The Oued Agrioun Development is but the first step in the Algerian plan of development, and will cover about 25 per cent. of the country's requirements. It is to be supplemented at a later date by the Oued Djen-Djen and Oued Bou-Sellam developments. (Abstract of an article by H. Castelnau and Y. Haller, *La Technique Moderne—Construction*, Vol. 9, No. 9, September 1954 p. 307, 15 pp., 32 ff.)

Problems of Hydro-Electric Design in Mixed Thermal and Hydro Systems

An important paper under this title was given last month before a joint meeting of the Institutions of Civil and of Electrical Engineers by T. G. N. Haldane, M.A., M.I.C.E., Past President I.E.E., and P. L. Blackstone, T.D., M.A., M.I.E.E.

THE paper referred to above was a timely one because it dealt with a subject that is engaging the earnest attention of power-supply engineers throughout the world. Actually the paper was concerned primarily with the integration of low-load-factor hydro-electric plant into a predominantly thermal system, but the authors also had interesting comments to make on the impact of nuclear power, the effect of rising prices, the design of hydro-electric plant generally, pumped storage, and electrical transmission.

After emphasising the complementary character of thermal and hydro power the authors refer to two well-known concepts—the annual load/duration curve, and the annual load factor. Seeing that the available energy for any site is virtually fixed by the run-off, a change in load factor usually means a change in installed capacity. This change does not materially affect the capital cost of storage, which forms a large proportion of the total cost, but it does affect the expenditure on tunnels, plant, buildings and transmission. Thus the incremental cost of increasing the capacity of a hydro station is usually less than that for a thermal station. For this and other reasons it is usually desirable to install hydro plant with the greatest installed capacity, i.e., the lowest firm load factor, that the system can accept.

In a purely thermal system the machines would be brought into operation sequentially in ascending order of their fuel cost per kilowatt hour, but it is not possible to achieve the theoretical optimum use of plant in practice, and a situation such as that shown diagrammatically in Fig. 1 supervenes. Here a typical annual load/duration curve has been drawn and on it have been superimposed the blocks of energy generated by different plants grouped in order of fuel cost per kWh. It will be seen that the more-efficient plants generate less than their theoretically desirable quota whereas the less-efficient plants generate more. This is shown in integrated form in Fig. 2, where curve (*a*) corresponds to the ideal load/duration curve and curve (*b*) represents what is actually achieved in practice, as indicated by the rectangles in Fig. 1.

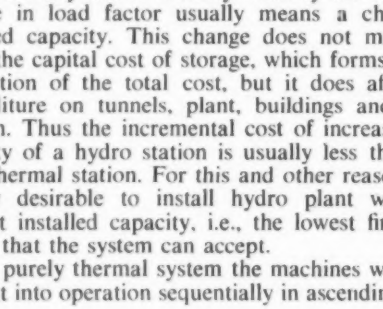


Fig. 1. Typical annual load/duration curve, showing energy generated by different blocks of plant

For determining the maximum practical installed capacity of a hydro-electric project the conventional annual, or even monthly, load/duration curves are inadequate, and it is necessary to consider daily duration curves. In Fig. 3, P represents the day of annual maximum demand and the remaining curves represent other typical days of the year. The normal type of annual duration curve is produced by integrating the areas of the individual daily curves enclosed by a series of horizontal lines such as AA and BB. Only on days P and Q does any load fall within these limits, and if a duration curve produced in this way were used as a guide to system operation the conclusion would be that some plant ought to run on those days only and be shut down on all other days of the year. This is not possible in practice and hence is misleading as a basis for judging results or forecasting the future.

There is no definite criterion for deciding how much energy the plant which generates a particular element of load on the day of maximum demand should generate on other days, but integration between curves such as (*a*) and (*b*) is regarded by the authors as a reasonable assumption. Fig. 4 shows a duration curve of the conventional type, and superimposed is the adjusted curve of the type obtained by integrating between the curved lines in the manner illustrated by

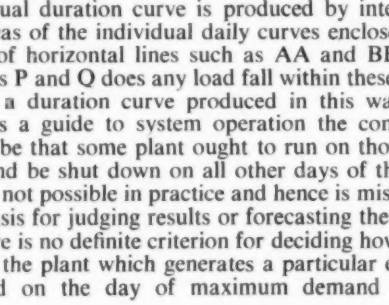


Fig. 2. Integrated duration curves

Fig. 3. The importance of the adjusted curve in Fig. 4 is that it is believed to provide a more realistic starting point in assessing the lowest acceptable firm annual load factor (or the maximum acceptable installed capacity) for any given block of energy.

Hydro-electric plant has a higher availability than thermal plant, even though this is reduced to some

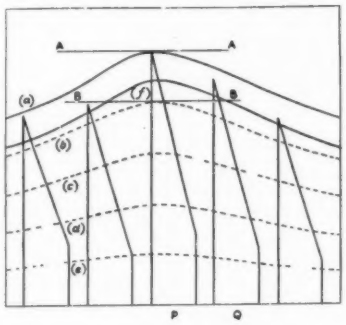


Fig. 3. Basis of adjusted load/duration curve: (a) Maximum demand (m.d.); (b) 80 per cent. m.d.; (c) 60 per cent. m.d.; (d) 40 per cent. m.d.; (e) 20 per cent. m.d.; (f) Minimum demand

extent by the longer transmission usually involved. It is more flexible because it can be started, synchronised and brought up to full load much more quickly. It is therefore especially suited to rapid load fluctuations, and its use in this way benefits the system as a whole. In wet weather, power can be generated at a load factor higher than that designed, thus saving fuel. Consequently, in a combined system, temporary departures from design operation will be desirable and will result in overall economy, whereas any departures in the all-thermal case produce an overall loss.

The introduction of hydro-electric plant on the top of the load/duration curve has four important effects on the total fuel consumption of the system.

(*a*) The fuel that would have been needed to generate the actual hydro-electric output is saved.

(*b*) Fuel for banking to keep the equivalent thermal plant available is saved.

(*c*) The quality of operation in the peak region is improved because the hydro-electric plant is capable of following more closely the fluctuations in demand.

(*d*) There is a temporary loss of the chance to improve the average thermal efficiency resulting from the addition of new thermal plant.

Factors (*a*), (*b*) and (*c*) will be seen to operate in favour of hydro-electric plant whereas factor (*d*) operates against it. The correctness of factor (*d*) was challenged in the discussion by one or two speakers; Mr. Tewson, for instance, pointed out that hydro-electric plant, by replacing inefficient peak-load thermal plant, might actually improve the overall thermal efficiency.

The authors then point out the dependence of firm hydro power on reservoir capacity and refer to the progressive increase—probably to a prohibitive amount—in the incremental cost of providing the highest percentages of firm power. Where storage is limited this may, in effect, mean that a certain acceptable load factor can be guaranteed throughout part of the year but not all of the time. Either the doubtful period must be eliminated by reduced installed capacity, or some means of firming must be provided. Two such means are possible, namely the installation of thermal plant especially for firming the water power, and the use of pumps. Either method can be regarded as a means of augmenting the effective capacity of a reser-

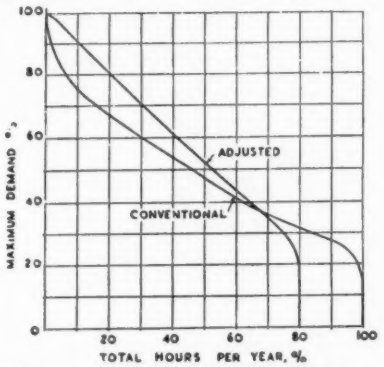


Fig. 4. Conventional and adjusted load/duration curves

voir, thus increasing the percentage of firm power, or as a means whereby the reservoir capacity can be reduced for a particular amount of firm power.

The use of thermal plant for firming is expensive, but may be justified; the use of storage pumps is a relatively new development, but considerable attention is now being given to this possibility, particularly in America. An energy loss of some 33 per cent. will be incurred, but pumping may be infrequent and the total amount of energy involved comparatively small. Moreover, the cost per unit of energy for pumping supplied at off-peak hours will be less than the value per unit of the stored energy if resold in peak hours.

Great emphasis is placed in the paper on the potentialities of pumped storage, and estimated data for three schemes are given in an appendix.

The possible impact of nuclear power on hydro-electric power is then examined, for although nuclear stations may not become commercially significant for a good many years, they may become so before hydro plants now being built become obsolete. The first British experimental nuclear power station will be in operation within two years, and the overall cost of generation is expected to be of the order of 1d. per kWh. Such stations will probably have the following characteristics:—

(*a*) For safety reasons they are likely to be placed in remote districts and will require long transmission lines. In this respect they are similar to most hydro stations.

(*b*) They are likely to involve relatively high capital costs and low running costs, and again there is some similarity with hydro plants.

(*c*) For technical reasons they will probably require to be kept under fairly constant load, and in this respect will differ from hydro stations.

As regards (*c*), in fact, the authors consider that nuclear stations will need to run as near 100 per cent. load factor as possible. On the other hand, Mr. F. J. Lane of the B.E.A., thought we could not expect a load factor of more than 75 per cent. in 1965, and that by 1985, owing to the increasing preponderance

(Continued on page 75)

Cavitation Testing of Francis Runners

Curves and photographs are presented to show the type of work being done in the Cavitation Laboratory of the Dominion Engineering Company, Limited.

By R. S. SPROULE, Manager of Hydraulic Division, and W. S. McILLQUHAM, Hydraulic Engineer.

IN this article the following glossary of terms applies to all the figures:

$\sigma = (H_B - H_S - H_V)/H$

where H_B = Barometric pressure.

H_S = Height of runner throat above tailwater.

H_V = Vapour pressure of water at corresponding temperature.

H = Net effective head on the turbine.

(σ is a comparison between the pressure at the runner throat and the available head. It is generally used as a criterion for the danger of damage or loss in performance due to cavitation. It has not, however, been fully established that performance of model and prototype are directly comparable on a sigma basis, without reference to scale or head.)

HP_{11} = Turbine power, reduced to horsepower for a one-foot diameter runner under one-foot head.

Q_{11} = Turbine discharge, cubic feet per second for a one-foot diameter runner under one-foot head.

ϕ = Speed ratio, the ratio between the peripheral speed of the runner throat and the theoretical spouting velocity of the water.

Figs. 1, 2 and 3 show how a 16 in. diameter model runner is tested over a range of sigma to determine its limitations with respect to setting above tailwater. Figs. 4 and 5 show how the performance of a runner may be visually inspected by means of a transparent draught-tube section. Figs. 6, 7 and 8 show how a slight modification, suggested by visual inspection, can make an important change in cavitation characteristics.

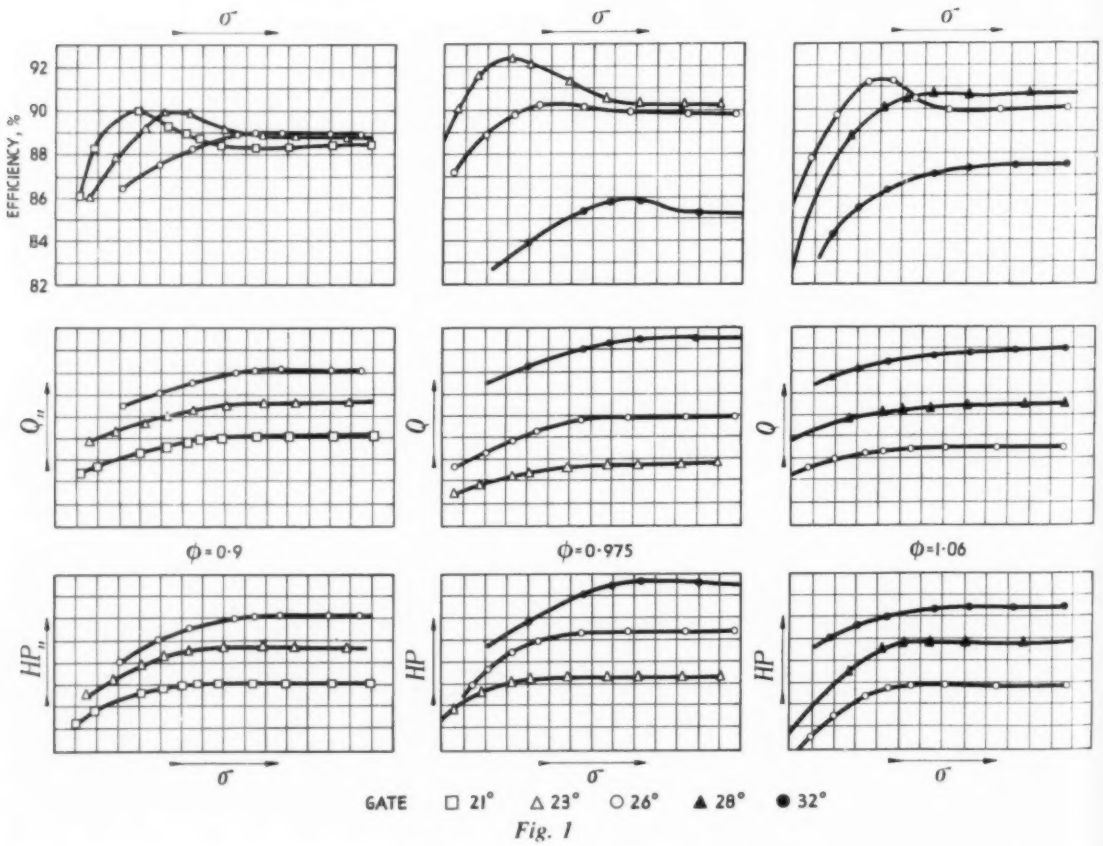


Fig. 1

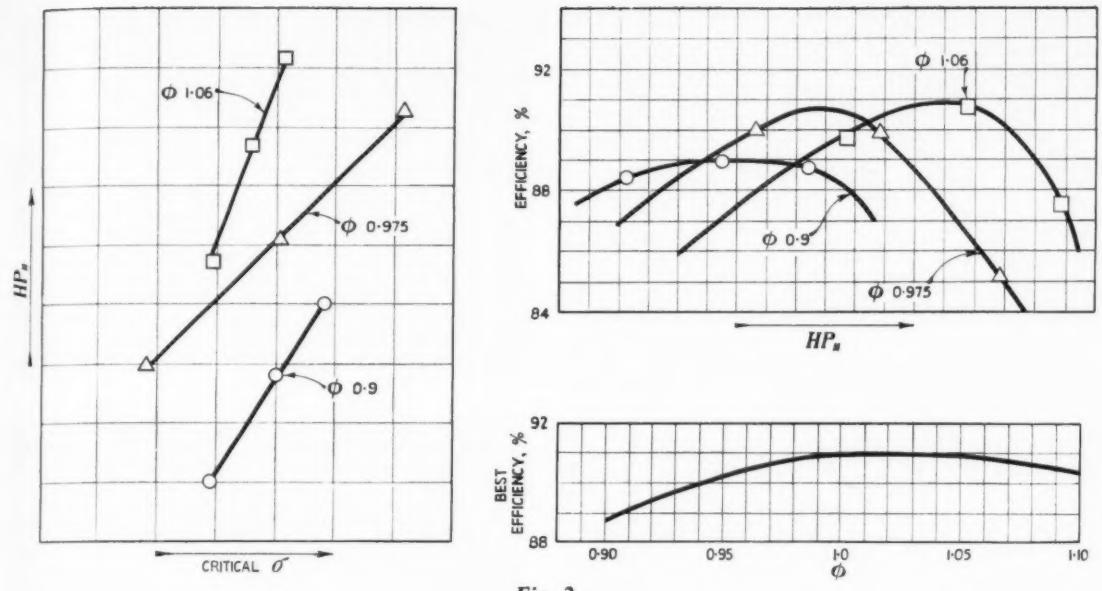


Fig. 2

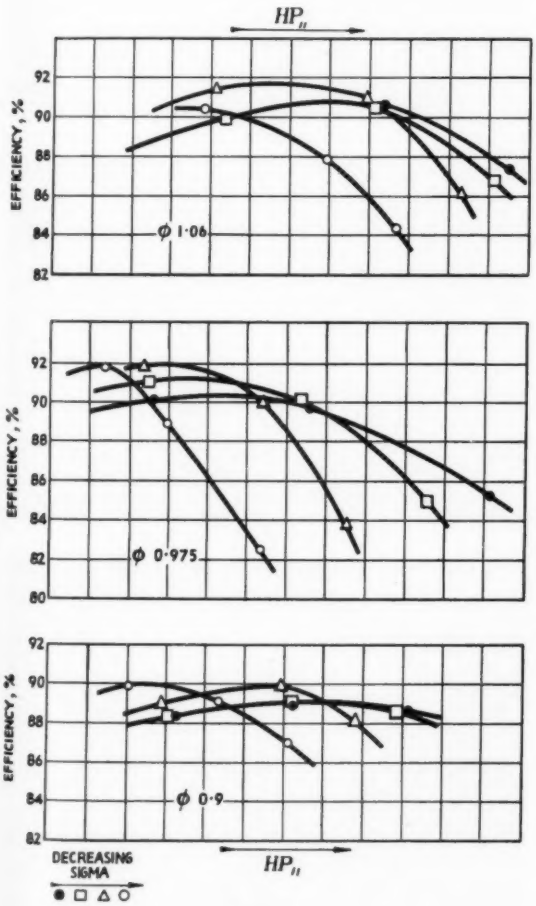


Fig. 3

Fig. 1 is a chart of power, quantity, and efficiency versus sigma for a high-specific-speed Francis runner, plotted at three different speed ratios and for three gate openings at each speed. Notice that the power at any gate and speed is constant as sigma is reduced, until a point is reached where it falls off rapidly with further reductions in sigma, indicating that flow over the blades is disturbed by the formation of cavitation bubbles.

The same is true of efficiency and quantity, except that usually the efficiency and sometimes the quantity are observed to increase over a range of sigma, before dropping off. The increase of efficiency noticed with this runner at the sigma where power starts to decrease has been observed on many runner tests. It does not usually occur at the point of best speed and power for the runner. (Notice the curve for 28 degree gate angle and $\phi = 1{\cdot}06$.)

In order to make comparisons, the authors have arbitrarily chosen a "critical sigma" as being the value of the cavitation parameter, at a given gate and speed, where the power is 1 per cent. less than at high values of sigma. This is employed in Fig. 2. Values of critical sigma taken from Fig. 1 have been plotted against power on the left side of Fig. 2 at three different speeds. The curves on the right side of the figure show how these speeds are related to the best efficiency speed for the runner. It will be noticed that for a given power the critical sigma is lower at a higher speed. In other words, less cavitation trouble would be expected from this runner, at a given power and setting, at high speed; or, for a given power, the turbine could be set higher if run at a higher speed. The authors have not yet sufficient data to say whether this is a general condition for Francis runners; possibly the reverse would be noticed at speeds considerably above the best efficiency.

Fig. 3 is a different chart of the same data as are shown in Fig. 1. It shows the very marked decrease in full gate power as sigma is reduced. This explains why a manufacturer must allow a generous margin

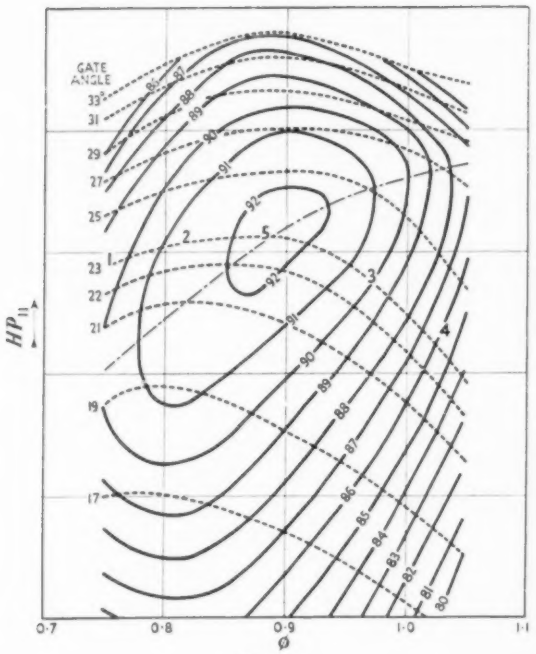
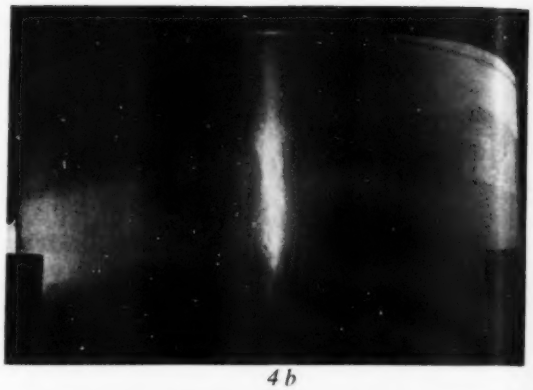


Fig. 4

of power over the guarantee if the setting is dangerously high. Notice that the efficiency and power near the best efficiency point are not seriously affected until a very low value of sigma is reached. The runner of Figs. 1, 2 and 3 is operating in the field at a setting equivalent to the critical sigma at the corresponding speed. Some cavitation repair is necessary to the mild-steel runner castings, but the performance is considered satisfactory.

Fig. 4 shows an efficiency hill chart for a Francis runner with photographs *a* to *d* showing the appearance of the draught tube immediately below the runner at various speeds with the gate opening which gives the peak efficiency, shown by the numbers 1 to 5 on the hill chart. Photograph *e* is a stroboscopic picture of the runner at the best efficiency point. There is no central core at the best efficiency. The pictures were taken below the critical sigma but the phenomena shown occur, to a smaller extent, at higher sigma.

Fig. 5 shows the corresponding phenomena at dif-



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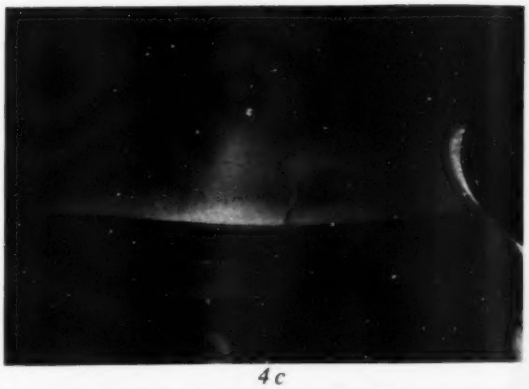

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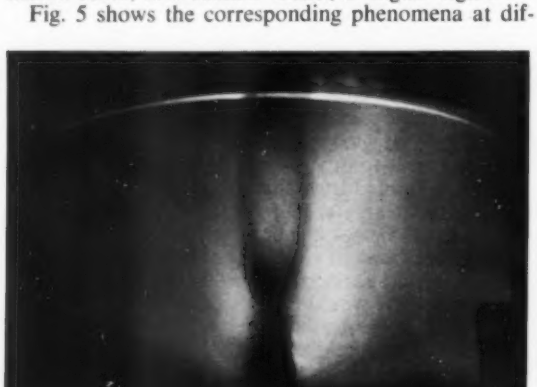

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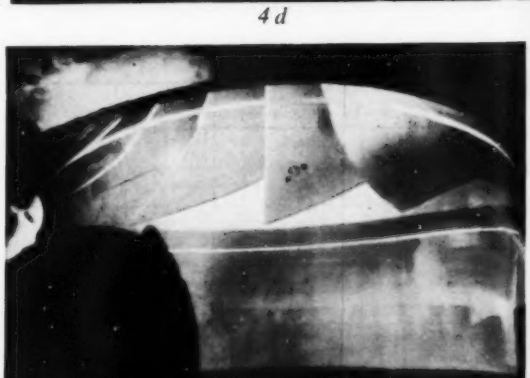

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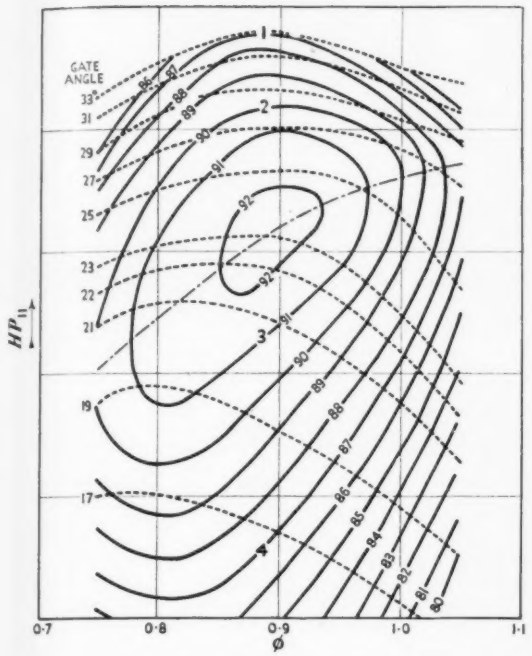


Fig. 5

ferent gate openings at the speed for best efficiency. A study of voids such as are shown in Figs. 4 and 5 is important in runner design, as they are indicative of the whirl in the water coming from different parts of the runner. The type of void shown by photograph *a* of both figures is due to strong negative water whirl, or whirl opposite to the rotation of the runner. It is usually stable. On the other hand the type of whirl shown by photograph *d* of both figures results from a positive whirl. This type of void is sometimes unstable and repeated collapse can cause heavy pounding in the draught tube.

Fig. 6 is a stroboscopic picture of a Francis runner taken at the best efficiency point and close to the critical sigma. The white patch on the back of blade number 7 is a bubble of cavitation forming behind the leading edge of the blade near the junction with the runner band.

Fig. 7 is a chart of this runner performance, at best efficiency gate, before and after making a modification suggested by Fig. 6. The modification was a properly shaped fillet as shown at *B* on Fig. 7. It is interesting to note that, although the performance was not measurably changed at high sigma, the critical sigma was drastically reduced.

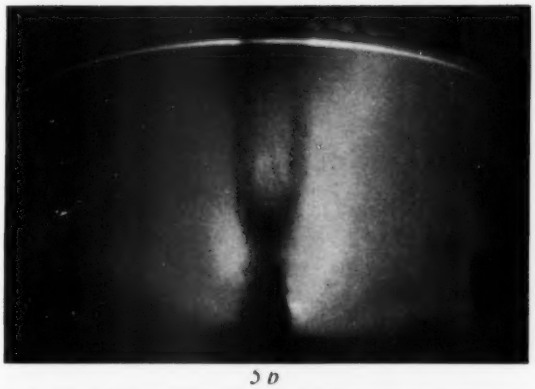
Fig. 8 is a similar chart for this runner at a gate opening which, at higher speeds, roughly corresponds to rated power.

Conclusions

The small modification to this runner allows a recommended turbine setting five or six feet higher than with the original shape. This is one example of the benefits which may be derived from running a well-equipped cavitation laboratory.

The Dominion Cavitation Laboratory enables the company's engineers to develop and predict perform-

5 a



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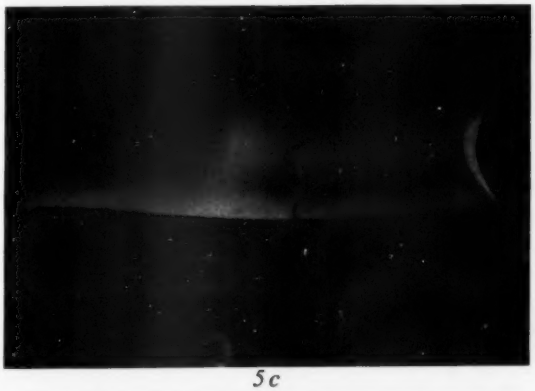

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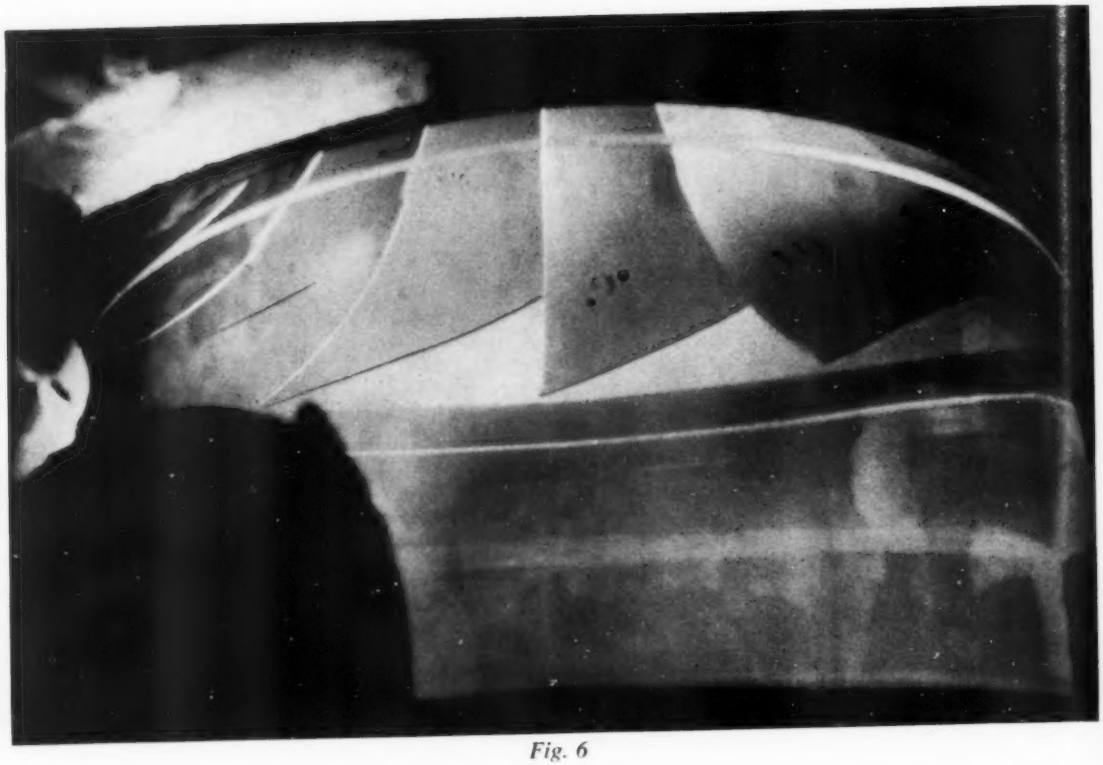


Fig. 6

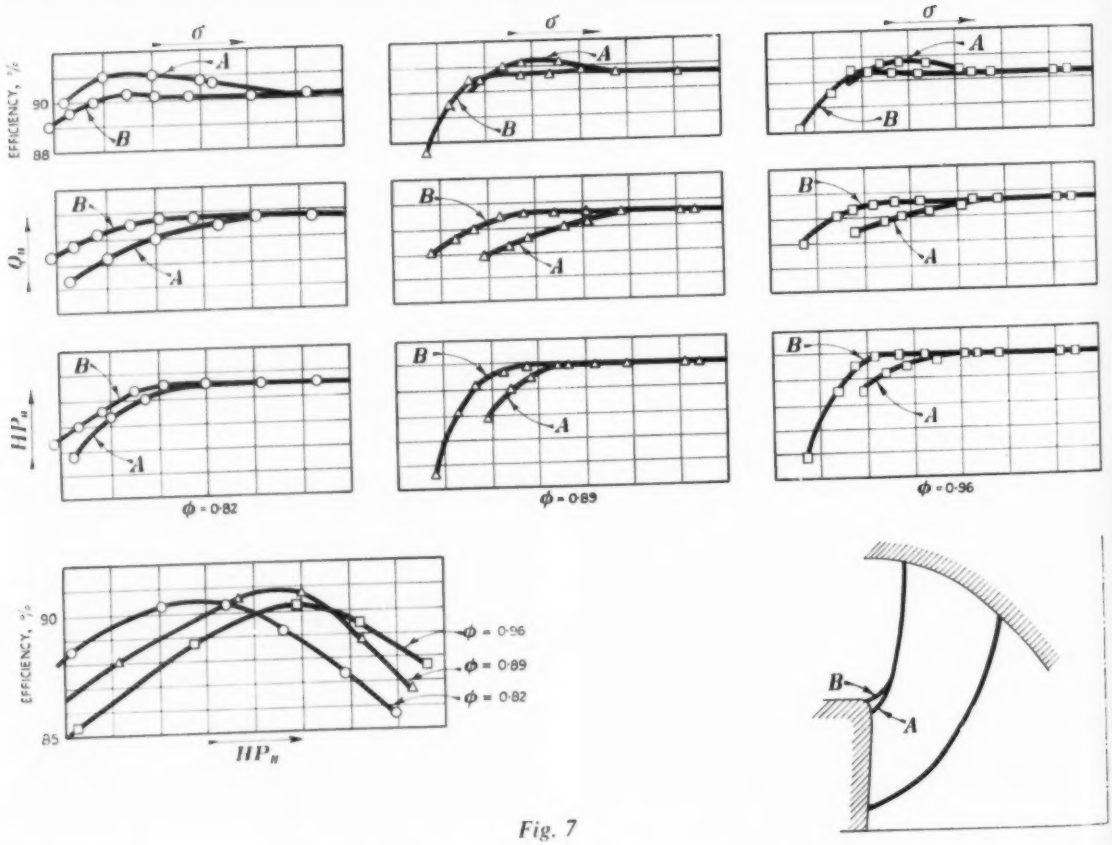


Fig. 7

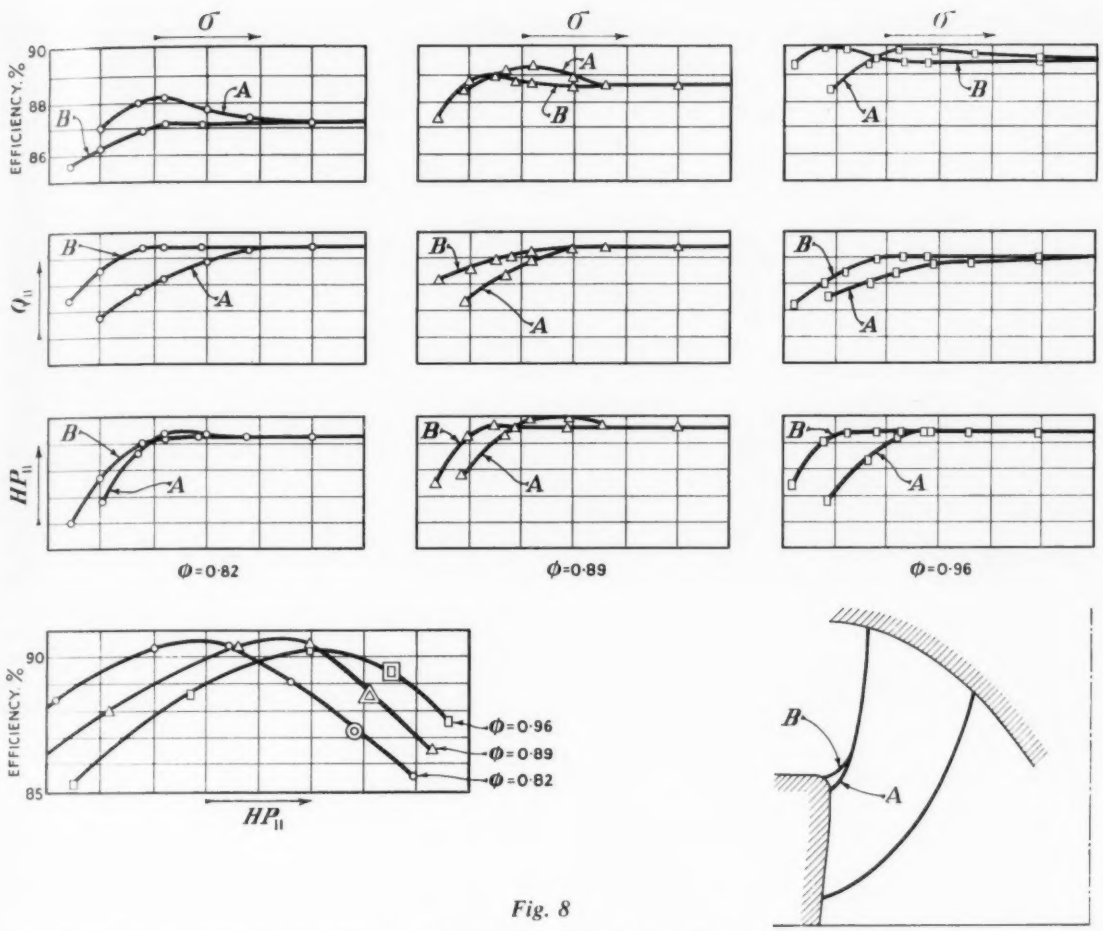


Fig. 8

ance of turbines running at economically high speeds and with a reasonable setting with respect to tailwater level. The cavitation laboratory is also used in the company's systematic efficiency testing of turbine runners.

The authors wish to express their thanks to Mr. H. S. Van Patter for his guidance in connection with this paper.

Ref. 1: "The design and calibration of a new Cavitation Laboratory for Hydraulic Turbines," by Hans Ulmann and R. S. Sproule, published in *Mechanical Engineering*, March 1953.

From page 69

of nuclear power, the load factor on these stations might fall to 62·5 per cent. In the discussion, also, Mr. Tewson considered that (*a*) would only be a passing phase, because nuclear plants would eventually be found safe enough to place near the load centres.

The authors then consider the probable effect of rising prices, and present an intriguing diagram intended to show that price trends will probably favour hydro plants on account of their higher initial cost and longer life. Present trends in the design of turbines, generators, reversible pump-turbines, control, and transmission are then surveyed.

Methods for increasing the capacity of transmission lines are mainly directed towards compensation of the line reactance and capacitance and the maintenance of transient stability, and include (a) High-speed switching, (b) Intermediate switching stations in double or multiple-circuit lines, (c) Rapid control of generator excitation, (d) Shunt compensating devices in the form of synchronous condensers or reactors, (e) Series capacitors. The collective effect of all the foregoing methods of improving a.c. transmission is so considerable that, for the present at least, there is no great urgency to develop d.c. transmission in those areas where a.c. overhead lines can be constructed. D.C. transmission offers the possibility of transmitting very large blocks of load over distances beyond the likely capacity of a.c. transmission, even at much higher voltages than at present in use. Such transmission may therefore become of importance in the future, although the prospect of this for overhead routes now seems more remote than at one time was thought possible. It seems likely that d.c. transmission will in the first instance be developed for submarine or underground routes, where its advantages are so outstanding. One such case is the proposed link between Great Britain and the Continent which, although likely to be an a.c. system initially, may later be greatly increased, and possibly cheapened, by the use of d.c. cables.

A Pioneer Pumped-Storage Scheme

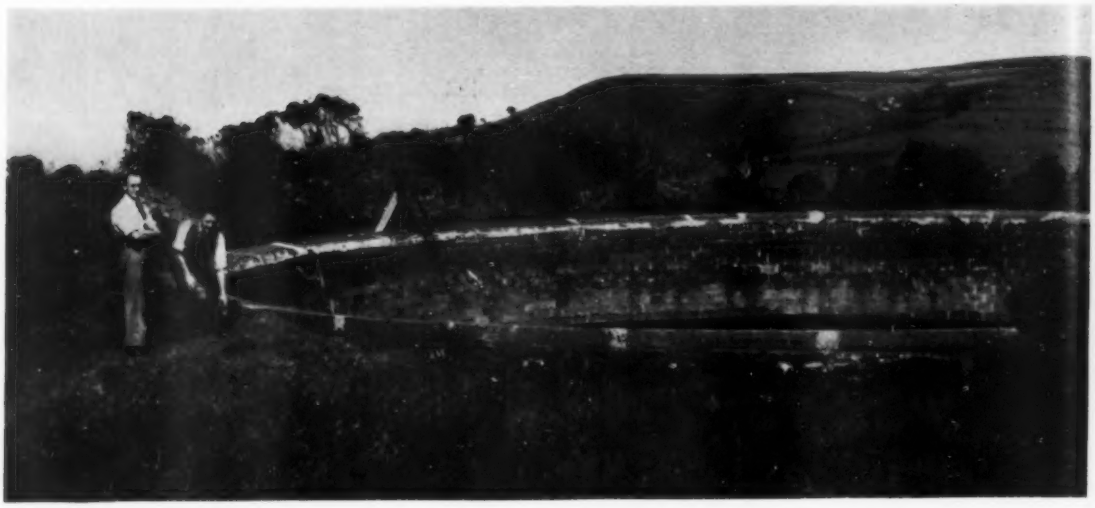


Fig. 1. The reservoir above Lynmouth into which the Bailey pump delivered water for subsequent use in the 50 h.p. Pelton wheel

We received recently, through the courtesy of Mr. A. C. Thirtle, Divisional Controller of the South Western Division of the British Electricity Authority, some interesting details about what must be one of the first pumped-storage schemes ever to be installed. It relates to the hydro-electric station established at Lynmouth in 1896.

Electricity supply commenced in Lynmouth in April, 1890, and it is believed to have been the second hydro-electric station established in Great Britain. By 1896 the load on the station was becoming greater than the plant could supply, and moreover the owner of a flour mill some miles upstream persistently interrupted the water supply at the period when the demand was heaviest. The company operating the power station therefore decided to install a pumped-storage scheme. During periods of low demand, surplus electrical energy generated by the water turbine was used to operate a pump made by Bailey's (the predecessors of Sir W. H. Bailey & Co. Ltd.) which lifted the water through a pipeline to a storage reservoir at Summer Hill, some 800 ft. above Lynmouth. The reservoir capacity was 190,000 gallons, and as it still exists it may be stated, in the present tense, that it is 50 ft. in diameter with a depth of 16 ft. 6 in. at the outlet side. When the peak demand developed, the water in the reservoir was fed back through the pipeline to operate a 50 h.p. Pelton turbine.

This pumped-storage system, which was installed so long ago, continued to operate until as recently as August, 1952, when the station was destroyed by the tragic floods of that period.

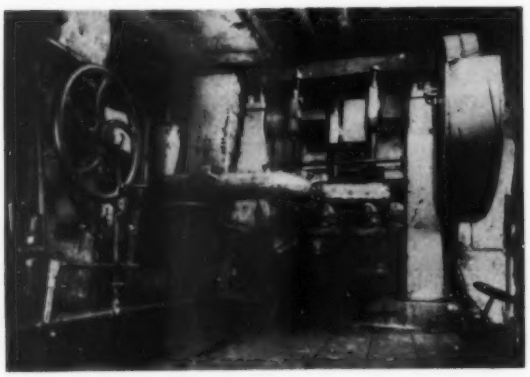
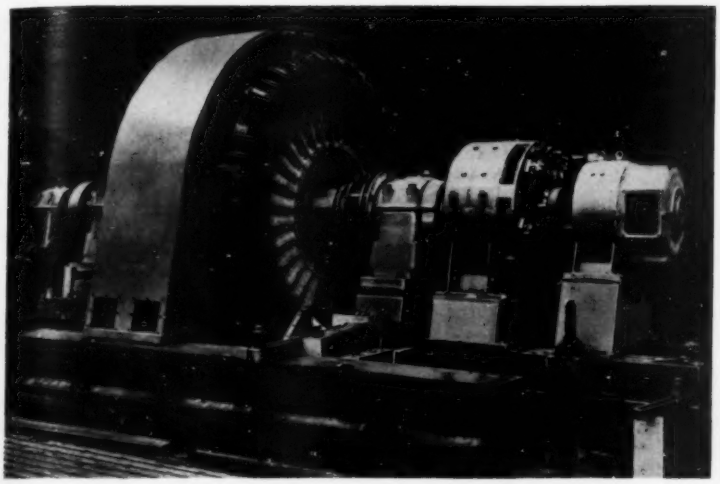


Fig. 2. The Bailey pump installed in 1896 for the Lynmouth pumped-storage scheme

Waterwheel Generator at Somerset Dam

Somerset Dam, in Queensland, Australia, a mass gravity structure providing a storage capacity of 724,000 acre-feet, was built by the Stanley River Works Board primarily to provide water for Brisbane 80 miles away and to mitigate floods in the Brisbane river valley. It was decided to install hydro-electric plant also so as to generate electricity from the water released, and the dam was provided with a 9 ft. diameter steel-lined tunnel leading to a power station containing a 3,200 kW 375 r.p.m. water turbine supplied by Boving & Co. Ltd. A double-runner turbine of the Francis type is coupled to a 4,000 kVA 0·8 power factor waterwheel generator, which with the switch and control gear was supplied by Metropolitan-Vickers Electrical Co. Ltd.

The generator, seen on the testbed in the accompanying illustration, is a standard horizontal machine with closed ventilation. The generating set can be controlled either locally or from a substation above the flood level 120 yards away. Remote control is semi-



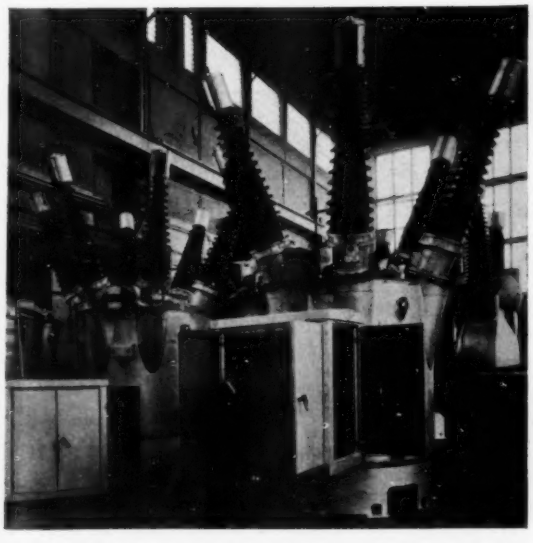
Waterwheel alternator for Somerset dam on the testbed in the Metropolitan-Vickers works

automatic: on operating the "start" switch the machine starts and runs up to speed, synchronising adjustments being carried out and the oil circuit-breaker being closed by operating the appropriate control switch. Local control can also be semi-automatic if required.

The hydro-electric power generated—the amount of course depends on the seasonal conditions—is stepped up from 11 to 33 kV and fed into the ring main of the Southern Electricity Authority at an adjacent substation. The power station attendant is in radio communication with the S.E.A. central control room.

Circuit-Breakers for Brechin

The accompanying illustration shows the 132 kV, 1500 MVA Shuntarc breakers, having all three phases in one tank, in course of manufacture at the British Thomson-Houston Company's Willesden works. These oil circuit-breakers are being supplied to the North of Scotland Hydro-Electric Board for installation at Brechin. This installation will be the first in Great Britain to employ 132 kV oil circuit-breakers having all three phases housed in a single tank, and it will be apparent that this construction has many advantages as regards installation and maintenance.



BTH 132 kV 1,500 MVA circuit breakers for Brechin

Sheepbridge-Built Tower Cranes

Sheepbridge Equipment Ltd., a subsidiary of Sheepbridge Engineering Limited, will shortly be manufacturing Jules Weitz tower cranes, under a licence secured by the parent firm. These new mobile electric tower cranes can be quickly dismantled, transported and re-erected, and have an outstanding advantage of being able to "spot place" loads of concrete, bricks, or other building materials, from ground level to points many storeys high in a matter of seconds, enabling contractors to effect a substantial saving of time, labour and handling costs. This new type of crane, which origi-

Sheepbridge/Jules Weitz tower crane in service on a ten-storey building

nated in France, is in use in many parts of the world on housing estates, multi-storey buildings, dock installations, dam-construction schemes, and so on. One of these fast-working tower cranes is doing good work on four six-storey blocks of L.C.C. flats, now nearing completion at Laburnam Grove, Mitcham, Surrey, and our illustration shows a tower unit in operation on a ten-storey building at Sevres, France.

Abstracts from the World Technical Press

New 230-kV Line in Philippines

This line, designed in connection with the Agno River project, is to connect the seven hydro plants of this scheme (Ambuklao, Binga, Itogon, Tabu, Tayum, Kalipkip, and Lubas) with Central Luzon and Manila. These plants, with an aggregate capacity of 430,000 kW, will be remote from any established power-consuming centre, so that the transmission system will be a major aspect of the development. Its main features will be: (1) a design based on the direct lightening-stroke theory; (2) use of non-magnetic aluminium clamps throughout; (3) measures against fatigue failure of conductors and ground wires; (4) elimination of arcing horns and grading shields; (5) long spans (normal 1,200 ft., max. 1·21 miles); (6) elimination of transposition. The Ambuklao-Manila line will consist of 21·7 miles on double-circuit and 114·1 miles on single-circuit towers; all double-circuit towers and 5·6 miles of the single-circuit line will be over very mountainous and rugged terrain, where landslides are common. The other 108·5 miles of single-circuit construction (Fig. 1) will cross relatively flat country; double-circuit towers will, however, be used where the siting of towers is difficult. The type of tangent heavy-construction towers designed for 2,300 ft. spans and a maximum wind loading of 16 lb. per sq. ft. at 45° F. is shown in Fig. 2. The line requires 32,000 kVA for charging, at 220 kV. The Ambuklao generators have a rating of 27,800 kVA each and a 1·1 short-circuit ratio; that is to say, a line charging capacity of 24,464 kVA each. Normally, two generators will charge the line. The power transformers at Ambuklao and Manila, however, will have a gross rating of 126,500 kVA, and will take an excitation of about 3 per cent., or 3,800 kVA. By lowering the sending end voltage to 206,000, the charging kVA will be reduced to 28,100 kVA. Thus, one generator with the terminal voltage reduced to 12 kV can charge the line, provided the receiving end transformers are connected to the line, as normally they will be. (A. C. Jimenez, *Electrical World*, Vol. 142, No. 9, 1954, p. 19, 2½ pp., 4 ff.)

The Chambal Project

A brief description is given of the various features of this project together with details of the Gandhi Sagar power station. The 600-mile long Chambal river is one of the main tributaries of the Jumna into which it empties south-east of Etawa. As now formulated, the project covers the building of three dams, in stages, with a power station at each. The first stage is to include the Gandhi Sagar installation with a power-house erected at the toe of a dam between the spillway and the right bank of the river; preliminary designs provide for four 23,000 kW generating units. The second stage comprises, besides the addition of further generating units at Gandhi Sagar plant, a power station at the Chulia Falls below the Rana Pratap Sagar Dam. The third stage will consist of a dam and a power station situated about 10 miles up-

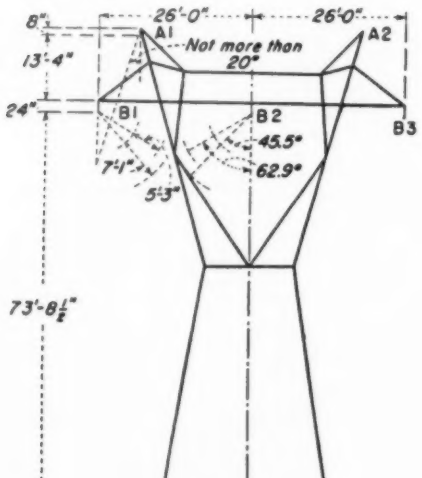


Fig. 1. Single-circuit tower for flat country

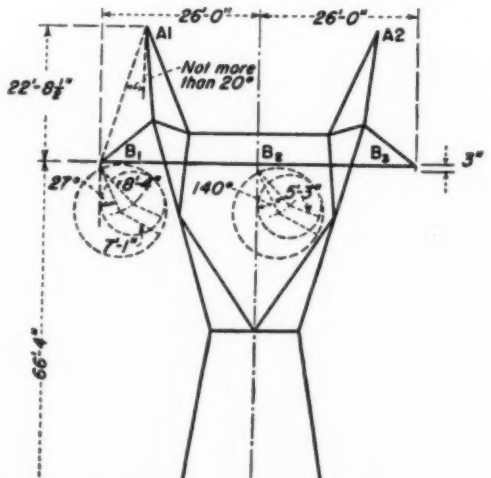


Fig. 2. Tangent heavy-construction tower

stream of Kotah. The multipurpose Chambal project is designed to transform parts of Madhya Barat and Rajasthan into rich agricultural tracts, and contribute to the industrial development of the region by means of cheap power. (*Power Engineer*, Simla, Vol. 4, No. 3, 1954, p. 125, 8 pp., 4 ff.)

Maggia Transformers

A description is given of the transformers supplied by the Oerlikon Engineering Company to the order of the Maggia-Kraftwerke A.G., Locarno. The Maggia group of power stations, which forms the first link of the 220 kV Swiss grid, comprises the underground Verbano and Cavergno plants and the Peccia plant, of which Verbano is already in operation and the two other plants in course of construction. The high-tension connections of the Maggia system are shown in Fig. 1. The main feature of the three-phase transformer banks, both at Verbano (two 64,000 kVA banks) and Cavergno (one 67,000 kVA bank), is that

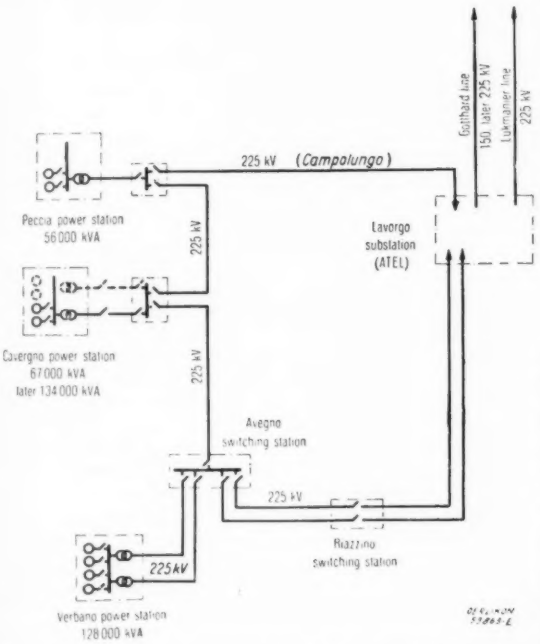


Fig. 1. High-voltage connections of the Maggia stations

they consist each of three single-phase units, delta-connected on the 12,000 V LT side, and star-connected on the HT side (maximum 251 kV). This arrangement greatly reduces transport difficulties, facilitates erection work underground, and has the substantial advantage of ensuring a reserve by the addition of a standby single-phase unit at each station. A thorough investigation of the various suitable types of winding, and the severe conditions to be met, led the manufacturers to select the arrangement shown in Fig. 2 for both plants; the high-tension winding, of the disc type subdivided by insulating collars, is not only highly stable but also secure against short-circuits, while low-tension windings are of the approved transposed helical type. On the low-tension side, the two legs are connected in series, and the high-tension side is con-

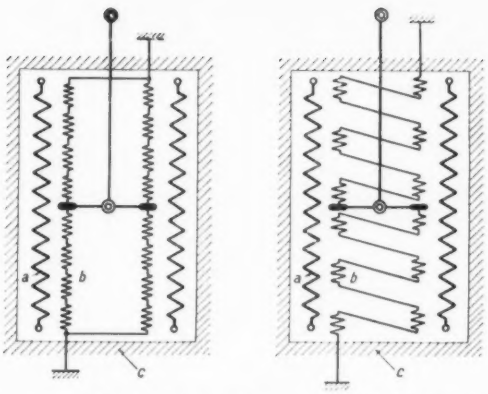


Fig. 2. Single-phase transformer with (left) *fourfold and* (right) *double parallel arrangement of the high-voltage winding*

nected as a twofold parallel circuit. The end spaces of each winding were designed to withstand the 100 kV test voltage of the high-tension side.

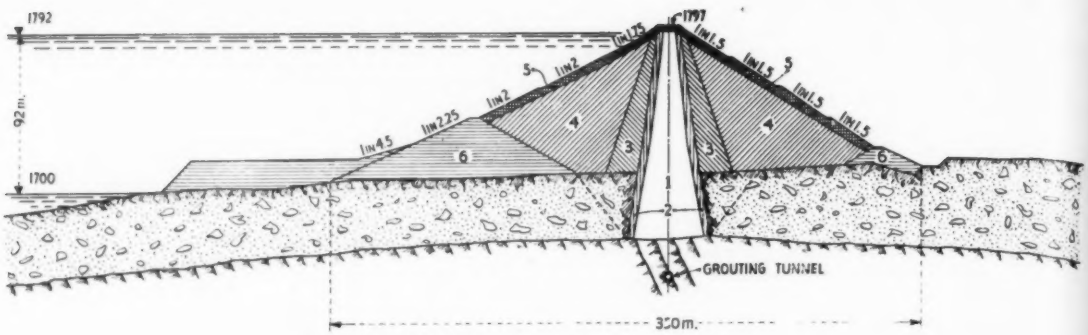
For transport, the transformers were filled with oil but stripped of all external components; the weight of each Verbano transformer in transit was 41 metric tons. The two Verbano banks, put into service in March and July 1953 respectively, have given every satisfaction in operation, and there have been no mishaps. (Hch. Lutz, *Bulletin Oerlikon (edition in English)*, Vol. XXXIV, No. 302, 1954, p. 1, 7 pp., 10 ff.)

Water-Power Byproducts

This article is based on a paper read by the author at the American Power Conference last April in Chicago, and describes the free benefits enjoyed by the public from the Big Creek Development, California. In addition to a production of 63,174 million kWh and a revenue of more than $5 million per year, these benefits are summarised as follows: (1) Supply to some of the nation's most productive lands of 70,000 acre-feet of irrigation water in an average water year and, upon completion of work now in progress, 120,000 acre-feet of new water in a drought year; (2) Flood control storage of about 200,000 acre-feet; (3) Four large playground lakes, five in another year; (4) Access to a playground of over 1,000 sq. miles. (W. L. Chadwick, *Engineering News-Record*, 1954, p. 38, 3 pp., 2 ff.)

Göschenalp Project

As far as we are aware this is the first major hydro-electric development in Switzerland in which the storage of power water will be provided by a rockfill dam. The erection of a gravity concrete structure was at first contemplated but, as was shown by preliminary estimates, the expenditure involved would have been prohibitive, and the project was eventually shelved as economically unrealisable; it re-emerged, however, when it became clear that the high-capacity earth-handling equipment imported by Swiss contractors from U.S.A. would enable them to set up a rockfill dam on the Göschenalp at about half the cost of a concrete dam. The design of the rockfill dam, shown in the appended diagram, was worked out on the basis



Cross section through Göschenalp rockfill dam

of extensive tests made at the Earthwork Laboratory of the Federal Technical University. The dam, 134 m. in height by 540 m. in length at the crest, 7·5 million cu m. in volume, will create a useful storage of 75 million cu. m. The natural catchment of no more than 42 sq. km. will be increased to 116 sq. km. by diverting the streams from neighbouring valleys. Half of the 10 km. access road was completed by the end of October 1954, and work on the dam itself was scheduled to be in progress by the end of 1954. (Dr. Ing. W. Eggenberger, Elektro-Watt A.G., Zurich, *Wasser-und Energiewirtschaft*, Vol. 46, 1954, p. 259, 7 pp., 8 ff.)

The Rhone and Upper Isère Plants

An account is given of an interesting tour of the Rhone and Upper Isère hydro-electric plants undertaken by the Civil Engineering Division of the Munich Technical University, under the leadership of its Vice-Chancellor, Professor A. Rucker. From some brief historical data, we learn that the first of all surge tanks, a stand pipe 34·7 m. in height, was erected at the downstream end of the penstock of the old power station at Champ, near Grenoble. (Dr. Ing. F. Franke, *Die Wasserwirtschaft*, Vol. 44, No. 8, 1954, p. 204, 4 pp., 8 ff.)

A 4,300,000-h.p. Project in N.W. Canada

Assuming the necessary licences are obtained from the Governments of Canada and British Columbia, work will begin in 1955 on a project which will rank among the major engineering developments of the world. The total watersheds to be developed extend 200 miles from the south end of Atlin Lake to the mouth of Big Salmon river, and a distance of 300 miles from Rancheria to Burwash. The water storage of the lake system in this area is, next to the Great Lakes, the largest in America; the minimum flow available is 28,000 cusecs. The storage will be at about 2,200 ft. elevation, and the fall of this water through tunnels to the Taku river will generate at least 4,300,000 h.p.—a potential about twice as great as the electrical energy to be developed by the St. Lawrence Seaway project. The main feature of the scheme is the diversion of the flow of the Upper Yukon river, and adjacent rivers and streams, to the valleys south of Atlin Lake; in fact, the 2,000-mile long Yukon river, which flows across Alaska into the Bering Sea, will be forced south to generate power through plants to be built in the Sloko, Nakonake and Taku river valleys, eventually emptying into the Pacific by way of the Taku river, south of Juneau, Alaska. Waters of the Big Salmon and Teslin rivers, tributaries of the Yukon, will also be used. At a later date, part of the upper waters of the Alsek river, which now flows into the Pacific, will be diverted into the Yukon river. The main purpose of the development is to supply power to metallurgical industries (cobalt, nickel, iron and steel, and manganese alloys) projected on a large scale in Northern B.C. and the Yukon. Ores and concentrates will be shipped to these plants from as far as New Caledonia, the Philippines and South-West Africa. (*The B.C. Professional Engineer*, Vol. 5, No. 8, 1954, p. 20, 4 pp., 1 map.)

Italian Electrical Industry

The report presented by the ANIDEL Council to its Associates in Rome on June 10, 1954, gives a full account of the activities of this Italian branch of the International Union of Producers and Distributors of Electrical Energy for the year 1953. An abundance of statistics, and maps and graphs in colour accompany this report which is followed, in an appendix, by the proceedings of the various Technical Committees of the Technological Board. An account is also given of the laboratory work performed at various universities and high technical schools of university rank. The proceedings of the No. 10 Committee, which ensures that the work of the hydraulic laboratories is properly co-ordinated, appear on page 623. (*L'Energia Elettrica*, Vol. XXXI, No. 8, 1954, p. 561, 69 pp., 28 tables, 16 plates of graphs and maps.)

CLASSIFIED ADVERTISEMENTS

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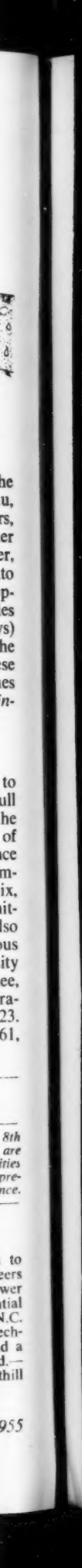
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